MILL CREEK REGIONAL

FACILITY PLAN REPORT

JCW NO. MCR1-BV-17-12 | BV PROJECT 403165

OCTOBER 2020

DRAFT

JOHNSON COUNTY KANSAS
Wastewater

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MILL CREEK REGIONAL FACILITY PLAN

Executive Summary

JCW NO. MCR1-BV-17-12
B&V PROJECT 403165

PREPARED FOR

JOHNSON COUNTY
KANSAS
Wastewater

OCTOBER 12, 2020

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1.0 Background and Objectives

The Mill Creek Regional (MCR) Wastewater Treatment Plant (WWTP) is located at 20001 West 47th Street, Shawnee, Kansas 66218 and primarily serves the Mill Creek, Tooley Creek, and Cedar Mill watersheds in Johnson County, Kansas. The original plant was constructed in 1995 as a mechanically aerated lagoon treatment facility. In 2006, the plant was expanded and upgraded to its current facility, which operates as two parallel treatment trains. The mechanical plant train is an activated sludge system sized to handle 12 million gallons per day (mgd) on an annual average (AA) basis (24 mgd peak flow). The lagoon train's current rated capacity is 6.75 mgd on an AA basis (84 mgd peak flow). A schematic of the current MCR WWTP is shown in Figure 1 with the wet weather flow split depicted. The red text in Figure 1 depicts flows to the lagoon train after an anticipated 12 mgd expansion of the Influent Pump Station (IPS), which will occur between the time of this Facility Plan and the MCR WWTP Expansion.

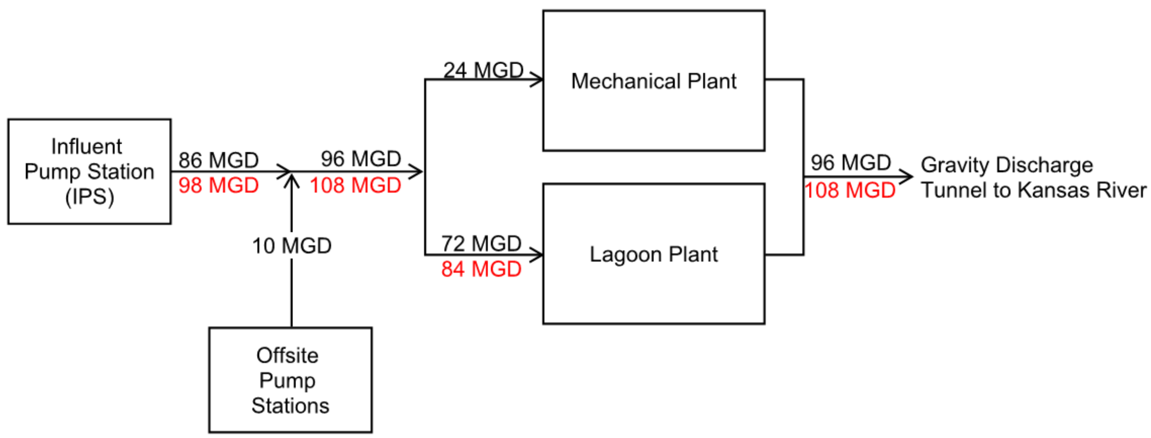


Figure 1 Current MCR WWTP Flow Schematic

The watersheds consist of a combination of mature suburban developments, new suburban developments, large commercial properties, business parks, and large plots of undeveloped land. Previous reports have estimated that, currently, the Mill Creek Watershed is approximately 60 percent developed. It is estimated that development will continue throughout the watershed until ultimate conditions are achieved.

The purpose of this Facility Plan is to recommend future improvements at the MCR WWTP to incorporate nutrient removal facilities, as required by the Kansas Department of Health and Environment (KDHE), to be implemented as part of an Integrated Management Plan (IMP). The findings inform JCW on the projected cost for the next MCR WWTP Expansion. This plan will be presented in Phase II of the IMP and allow resources to be arranged to meet the IMP's objectives.

2.0 Approach

The findings of this Facility Plan are presented across ten different technical memorandums (TMs). The first TM is focused on the design flows and loads, along with estimated future KDHE permit limits. TM 2 through TM 7 determine the plant processes starting with preliminary treatment through biosolids treatment and support facilities. Each TM provides a recommended treatment technology with associated layouts, capital costs, and operational and maintenance (O&M) costs. TM 8 focused on-site optimization and maintenance of plant operations (MOPOs) during construction of the MCR WWTP Expansion. TM 9 investigates the influent pumping at the MCR

WWTP, along with the pumping at offsite pump stations that send flow to the MCR WWTP. TM 10 presents implementation of the selected treatment technologies including permitting, scheduling, and total project costs.

It is important to note that, prior to this Facility Plan Report, an extensive treatment technology alternative analysis was completed for the JCW Tomahawk Creek (THC) WWTP Expansion project. Treatment technology evaluations consisted of utilizing a triple bottom line (TBL) approach to evaluate non-economic factors in addition to developing capital and operating costs for each alternative. Several of the selected treatment technologies from this analysis were also used in the planning of the MCR WWTP Expansion. These THC WWTP treatment technology evaluations are applicable for the MCR WWTP as THC is a similarly sized facility with similar wastewater characteristics, and both facilities are owned and operated by JCW. The recommended treatment facilities at the MCR WWTP largely align with the THC WWTP. However, there are exceptions where the ample site space at the MCR WWTP, operational factors, regulatory understanding, and future considerations resulted in different conclusions than those implemented at THC WWTP. Additionally, TBL evaluations specific to the MCR WWTP were completed for primary treatment (TM 2), disinfection (TM 5), and dewatering (TM 6).

3.0 Summary

3.1 FLOWS, LOADINGS, AND PERMIT LIMITS

After reviewing the MCR WWTP flow data over the last five years and estimating the future growth rate, it is recommended that the MCR WWTP Expansion project be sized for the ultimate growth conditions. The historical data analysis at the MCR WWTP concluded that the influent loading concentrations at MCR compare very similarly to JCW facilities at THC and the Blue River Main WWTP. Table 1 summarizes the MCR design flows and loadings.

Table 1 Design Flows and Loading Summary

PARAMETER	ANNUAL AVERAGE (AA)		MAXIMUM MONTHLY AVERAGE (MM)		PEAK DAY (PD)
Flow, mgd	21.0		31.5		126.0
	mg/L	ppd	mg/L	ppd	ppd
Biochemical Oxygen Demand (BOD)	207	36,200	179	47,100	72,400
Total Suspended Solids (TSS)	280	49,000	240	63,800	98,000
Volatile Suspended Solids (VSS)	247	43,200	211	56,200	86,200
Total Kjeldahl Nitrogen (TKN)	41	7,160	35	9,100	12,500
Ammonia	22	3,860	19	4,920	6,780
Total Phosphorus (TP)	4.8	840	4.2	1,100	1,480
Ortho-Phosphate	1.9	340	1.8	460	600

Nutrient removal goals of 10 mg/L for Total Nitrogen (TN) and 1.0 mg/L for Total Phosphorus (TP) as AA concentrations are included in the permit. Additionally, a limit of 156.63 pounds per day (ppd) TP as a 12-month rolling average is included in the plant's National Pollutant Discharge Elimination System (NPDES) permit. Compliance with these goals and limits is required one year after substantial completion of the MCR WWTP Expansion Project.

3.2 TREATMENT PROCESSES

The selected treatment processes are shown arranged in a Process Flow Diagram in Figure 2 at the end of this section. A brief summary of the selected treatment processes is summarized below. All processes are new except the Influent Pump Station.

Flows from the Mill Creek Interceptor arrive to the MCR WWTP at the existing Influent Pump Station. Flow is screened and wet-pit submersible pumps lift the flow to the Headworks Building. The Headworks building will house the fine screening and grit removal equipment. Fine screening equipment will include three channels with a shallow flow through perforated plate fine screen and a sluice trough with two washer-compactors. The grit removal system will include two free vortex Headcell units, two washer-dewatering units, and two slurry pumps. Primary Treatment is provided by four circular Primary Clarifiers.

The recommended secondary treatment process is a four-train plug flow biological nutrient removal (BNR) process arranged in the sidestream enhanced biological phosphorus removal (S2EBPR) configuration. Aeration demands will be met by five high-speed gearless turbo blowers. Four circular Final Clarifiers will be provided to clarify the BNR effluent.

Six Cloth Disk Filters will be provided to treat auxiliary wet weather flows exceeding peak secondary flow (3Q) and will be used for tertiary treatment during dry weather. Ultraviolet (UV) disinfection technology will be used for all flows.

Solids processing will be provided to produce a Class B biosolids cake suitable for land application. This will be achieved with mesophilic anaerobic digestion and centrifuge dewatering. The phosphorus recovery process is included to recover phosphorus from the centrate. Prior to digestion, waste activated sludge (WAS) will be thickened in two stages of dissolved air flotation (DAF) tanks to support the phosphorus recovery process. Primary sludge will be thickened in Gravity Thickener/Fermenter tanks. A sidestream deammonification system will be provided to treat effluent from the phosphorus recovery process for enhanced removal of ammonia and nitrogen.

3.3 SITE OPTIMIZATION AND MOPO

After selecting treatment processes, the facilities were oriented in a way that achieves several interdependent objectives as described below. The recommended site layout is shown in Figure 3.

- Provide an efficient facility layout from a wastewater operations and hydraulic perspective.
- Maintain plant operations during construction to meet permit limits.
- Provide redundancy to critical areas to eliminate single points of failure.
- Allow for constructability, and sequencing of future facilities, and identifying locations to allow a streamlined construction.

The recommended MOPO strategy includes using existing Cell 8 for wet weather treatment until the Filter Complex and UV Disinfection Building are constructed. The existing Cell 6 will be used for solids storage and processing until the new solids processing facilities are operational.

3.4 PROJECT SCHEDULE

JCW's IMP requires the MCR WWTP Expansion to be completed and operational by the year 2035. To achieve this milestone, it is recommended that the engineering design phase begins by March 2028. Engineering design will last approximately three years and will occur concurrently with the anticipated construction manager at risk (CMAR) pre-construction activities. The construction phase is anticipated to be approximately five years. A summary of several key project milestones is included in Table 2.

Table 2 Project Schedule Milestones

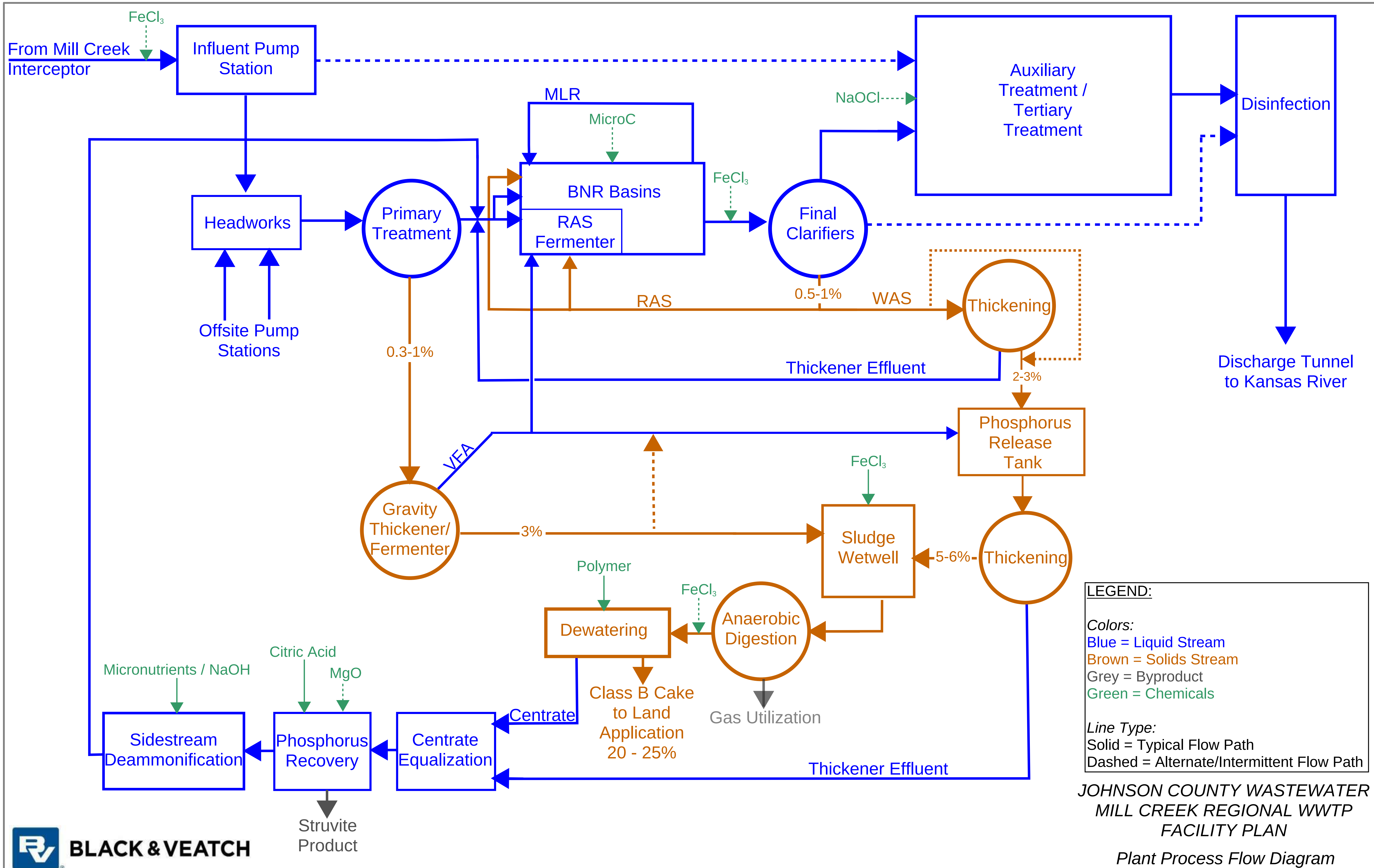
ACTIVITY	START DATE	END DATE	DURATION
Engineer Selection	08/2027	03/2028	9 months
CMAR Selection	03/2028	10/2028	8 months
Design	03/2028	11/2030	32 months
Construction Phase	12/2030	9/2035	57 months
Site Fill & MOPO	12/2030	12/2031	12 months
Startup & Substantial Completion	06/2033	12/2034	18 months
Demolition & Closeout	12/2034	09/2035	9 months

3.5 PROJECT COSTS

Planning level costs for future facilities at the MCR WWTP were primarily developed by adjusting the recent, similar THC facilities cost based on time and size. The opinion of probable project cost is presented in Table 3.

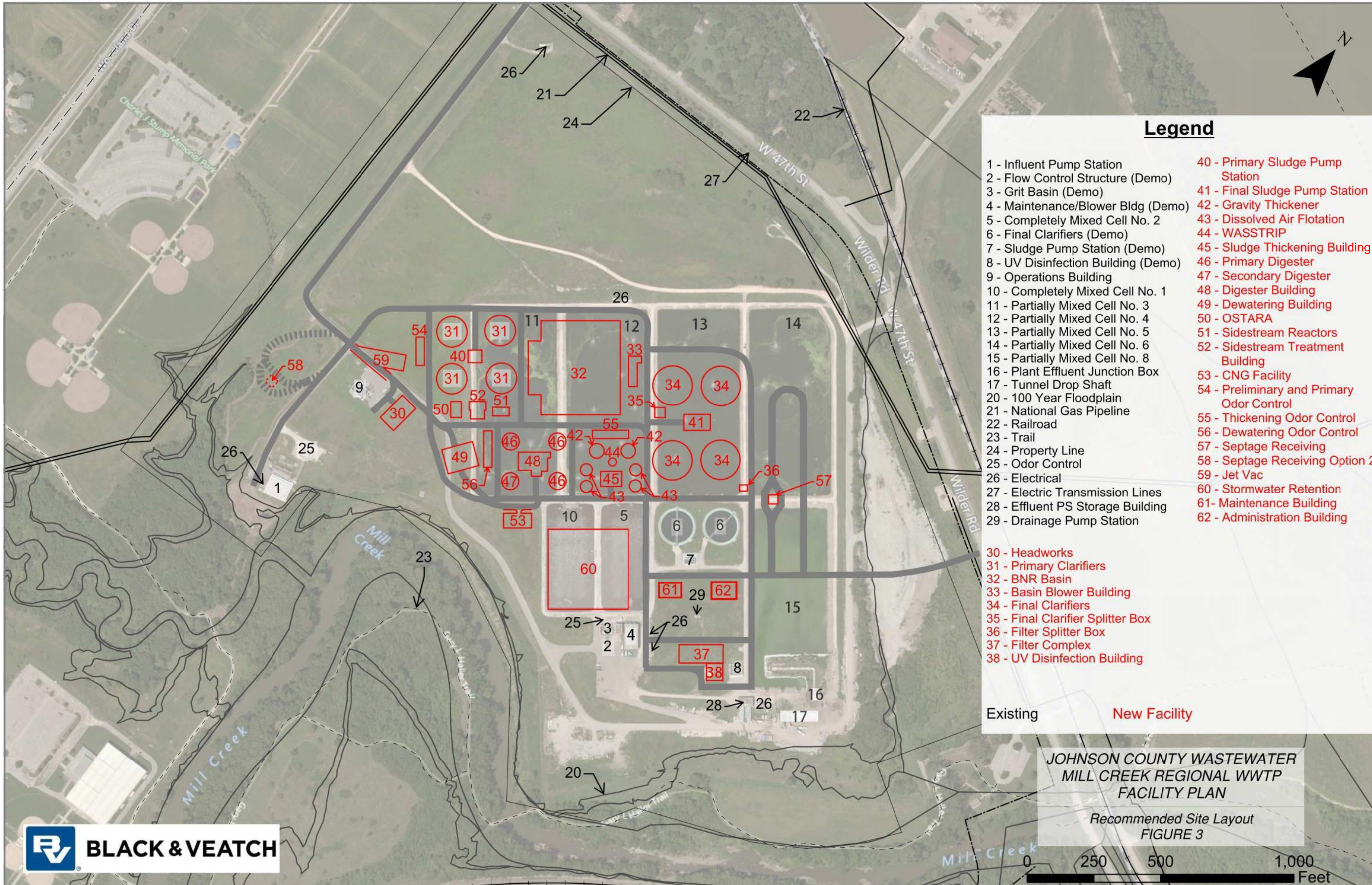
Table 3 Opinion of Probable Project Costs

	CAPITAL COST
Opinion of Probable Construction Cost / Projected CMAR Guaranteed Maximum Price (GMP)	**$378,900,000**
ELA – 18%	$68,202,000
JCW Administration Fee – 1.5%	$5,684,000
CMAR Pre-Construction Fee	$3,500,000
FFE, Utilities	$3,000,000
Opinion of Probable Project Cost (2020)	**$459,000,000**
Opinion of Probable Project Cost (2031)(1)	**$635,000,000**
(1) Future 2031 costs were escalated using 3% per year.	



JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

Plant Process Flow Diagram
FIGURE 2


Legend
1 - Influent Pump Station
2 - Flow Control Structure (Demo)
3 - Grit Basin (Demo)
4 - Maintenance/Blower Bldg (Demo)
5 - Completely Mixed Cell No. 2
6 - Final Clarifiers (Demo)
7 - Sludge Pump Station (Demo)
8 - UV Disinfection Building (Demo)
9 - Operations Building
10 - Completely Mixed Cell No. 1
11 - Partially Mixed Cell No. 3
12 - Partially Mixed Cell No. 4
13 - Partially Mixed Cell No. 5
14 - Partially Mixed Cell No. 6
15 - Partially Mixed Cell No. 8
16 - Plant Effluent Junction Box
17 - Tunnel Drop Shaft
20 - 100 Year Floodplain
21 - National Gas Pipeline
22 - Railroad
23 - Trail
24 - Property Line
25 - Odor Control
26 - Electrical
27 - Electric Transmission Lines
28 - Effluent PS Storage Building
29 - Drainage Pump Station
30 - Headworks
31 - Primary Clarifiers
32 - BNR Basin
33 - Basin Blower Building
34 - Final Clarifiers
35 - Final Clarifier Splitter Box
36 - Filter Splitter Box
37 - Filter Complex
38 - UV Disinfection Building
40 - Primary Sludge Pump Station
41 - Final Sludge Pump Station
42 - Gravity Thickener
43 - Dissolved Air Flotation
44 - WASSTRIP
45 - Sludge Thickening Building
46 - Primary Digester
47 - Secondary Digester
48 - Digester Building
49 - Dewatering Building
50 - OSTARA
51 - Sidestream Reactors
52 - Sidestream Treatment Building
53 - CNG Facility
54 - Preliminary and Primary Odor Control
55 - Thickening Odor Control
56 - Dewatering Odor Control
57 - Septage Receiving
58 - Septage Receiving Option 2
59 - Jet Vac
60 - Stormwater Retention
61- Maintenance Building
62 - Administration Building
Existing
New Facility
W 47th St
Wilder Rd
Mill Creek
Charles J Stump Memorial Park
N
JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN
Recommended Site Layout
FIGURE 3
0 250 500 1,000
Feet
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MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 1
Background, Flows, Loadings, and NPDES Limits

JCW NO. MCR1-BV-17-12
B&V PROJECT 403165

PREPARED FOR

JOHNSON COUNTY KANSAS
Wastewater

SEPTEMBER 17, 2020

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Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CANDO	Coupled Aerobic-anoxic Nitrous Decomposition Operation
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEA	Cost Effective Analyses
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
CIPP	Cured-in-place Pipe
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSBR	Continuous Sequencing Batch Reactor
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
D	
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry-weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand

Abbreviation	Meaning
ffCOD	Flocculated Filtered Chemical Oxygen Demand
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
ft	Feet
Fps	Feet per Second
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
gpcd	Gallons per capita per day
gpd	Gallons per Day
gpm	Gallons per minute
H	
HB	Hallbrook Facility
HDD	Horizontal Directional Drilling
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration
IC	Internal Combustion
IFAS	Integrated Fixed-Film Activated Sludge
in	Inches
IND	Industrial
INF	Influent
IP	Intellectual Property
IPS	Influent Pump Station
IR	Irrigation Use
IRR	Irrigation
IW	Industrial Water Supply Use
J	
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDHE	Kansas Department of Health and Environment
K_e	Light Extinction Coefficient
kWh	Kilowatt-Hour
L	
L	Length, Liter
lb	Pound
LF	Linear Feet
LOMR	Letter of Map Revision
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter
mgd	Million Gallons per Day
min	Minute, minimum
mJ	Millijoules
MLE	Modified Ludzack Ettinger

Abbreviation	Meaning
MLSS	Mixed Liquor Suspended Solids
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow
mmBtu	Million British Thermal Units
MOPO	Maintenance of Plant Operations
mpg	Miles per Gallon
MPN	Most Probable Number
µg/L	Micrograms per Liter
N	
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NFIP	National Flood Insurance Program
NH3-N	Total Ammonia
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
NPV	Net Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow
PE	Primary Effluent
PFE	Primary Filtered Effluent
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow

Abbreviation	Meaning
PIF	Peak Instantaneous Flow
PLC	Programmable Logic Controller
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day
pph	Pounds per Hour
PPI	Producer Price Index
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet-Weather Flow
Q	
Q	Flow
R	
RAS	Return Activated Sludge
RAS	
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
Sec	Second, Secondary
SF	Square Foot
SG	Specific Gravity
SLR	Solids Loading Rate
SMP	Stormwater Management Program, Shawnee Mission Park Pump Station
SND	Simultaneous Nitrification/ Denitrification

Abbreviation	Meaning
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station
SRT	Sludge Retention Time
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
TYP	Typical
U	
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VFA	Volatile Fatty Acids
VFAs	
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	Width
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation –Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WL	Water Level
WK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

1.1 FACILITY PLAN OBJECTIVES

The purpose of this project is to study future improvements at Mill Creek Regional (MCR) Wastewater Treatment Plant (WWTP) to incorporate nutrient removal facilities and processes as required by Kansas Department of Health and Environment (KDHE). The study will investigate the treatment technologies, footprint, operational, and economic impacts required to meet KDHE ammonia limits, total nitrogen, and total phosphorus goals. By January 2021, JCW shall submit to KDHE:

- An engineering study for nitrogen removal at MCR, in order to achieve compliance with the final ammonia limits, and the intent of meeting the effluent total nitrogen goal of 10 mg/L
- An engineering study for total phosphorus removal at MCR in order to achieve compliance with the final limits for total phosphorus of an annual rolling average load limit as required by the EPA approved Total Maximum Daily Load and Waste Load Allocation of 156.63 lbs/day, and with the intent of meeting the effluent total phosphorus goal of 1.0 mg/L. Once effective, the limit will be a rolling 12-month average calculated on a monthly basis.

This facility plan will inform JCW on the projected cost for the next MCR WWTP expansion. This plan will be presented in Phase II of the Integrated Plan and allow resources to be arranged to meet the Integrated Plan's stated objectives.

The Facility Plan will incorporate findings from 9 different technical memorandums (TMs) into a single report. The first TM will confirm dry and wet weather design flows and loads along with estimated future KDHE permit limits. TMs 2 through 7 will be focused on plant processes such as fine screening, grit removal, primary treatment, secondary and sidestream treatment, auxiliary treatment and disinfection, biosolids treatment, and support facilities. Each TM will provide a recommended treatment alternative that will be the basis for developing footprint, capital, and operational and maintenance costs. TMs 2 and 5 will consist of alternative development and evaluation to determine the preferred technologies for primary treatment and disinfection, respectively, to meet current and anticipated NPDES limits for design flows. TMs 8 and 9 will be focused on site optimization and implementation of all the selected treatment technologies including developing up to 3 facility layouts and hydraulic profiles showing major site pipe sizes and routing. As part of the alternative evaluations, opportunities to implement energy optimization strategies in accordance with the Owner's Triple Bottom Line (TBL) and Engineer's cost-benefit approach will be identified. Lastly, the total project costs and overall project schedule for implementation of the improvements including projected operation and maintenance (O&M) costs and staffing levels required will be refined and provided in the Final Facility Plan

1.2 BACKGROUND

The MCR WWTP is located at 20001 West 47th Street, Shawnee, Kansas 66218 and primarily serves the Mill Creek, Tooley Creek, and Cedar Mill watersheds in Johnson County, Kansas. The original plant was constructed in 1995 as a mechanically aerated lagoon treatment facility. In 2006, the plant was expanded and upgraded to its current facility, which operates as two parallel treatment trains. One train is an activated sludge system sized to handle flows up to approximately 12 mgd (on an annual average basis, 24 mgd peak flow) and the other is a lagoon system sized to handle flow in excess of what the activated sludge system can effectively treat. The current rated capacity of the lagoon train is 6.75 mgd (on an annual average basis, 84 mgd peak flow).

Wastewater from the Mill Creek watershed flows primarily by gravity through a sanitary sewer network to the Influent Pumping Station where it is screened and pumped to the flow control structure. At the flow control structure, influent from the Cedar Mill and Tooley Creek / 55th Street pumping stations combines with the flow from the Influent Pumping Station, and then flow is split between the mechanical plant and the lagoon train. Flow from the lagoon train and mechanical plant train recombine downstream of the UV building before flowing by gravity through the effluent tunnel to the Kansas River. Wet weather flow to the mechanical plant is limited to 24 mgd (two times rated annual average capacity or 2Q). A schematic of MCR is shown in Figure 1-1 with the wet weather flow split depicted. The red text in Figure 1-1 depicts flows the wet weather flow split after an anticipated 12 mgd expansion of the Influent Pump Station (IPS), which will occur between the time of this study and the time of plant expansion.

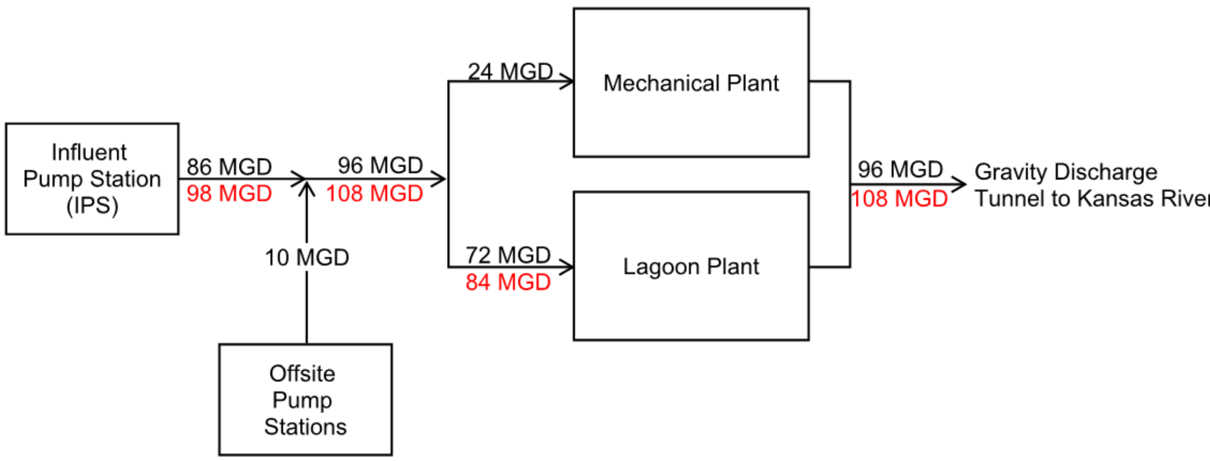

Figure 1-1 MCR WWTP Flow Schematic

The activated sludge (mechanical) train consists of a single completely mixed aeration cell (Cell 2), two final clarifiers, a sludge pumping station, and ultraviolet (UV) disinfection. The lagoon train contains one completely-mixed aeration cell (Cell 1) followed by five partially-mixed aeration cells (Cell 3, 4, 5, 6, and 8). Facilities shared by both treatment trains include an influent pumping station, flow control structure, forced vortex grit removal basins, maintenance/blower building, and gravity discharge effluent tunnel. A summary of both the mechanical and lagoon train existing treatment equipment is included below:

Mechanical Train:

- 1 Completely Mixed Aeration Cell with a total volume of 958,000 cubic feet (cf) and coarse-bubble diffusers with 20,200 cubic feet per minute (cfm) per cell
- 3 Single-Stage Centrifugal aeration blowers with 36,000 standard cubic feet per minute (scfm) firm capacity
- 2 Circular 130-foot diameter final clarifiers, each rated for 12 mgd
- 3 Horizontal end suction centrifugal return activated sludge (RAS) pumps each with 7 million gallons per day (mgd) capacity

- 3 Horizontal end suction centrifugal waste activated sludge (WAS) pumps each with 180 gallons per minute (gpm) capacity
- 1 Horizontal end suction centrifugal scum pump with 180 gpm capacity
- 1 UV Disinfection facility with 4 horizontal high intensity UV banks rated for 24 mgd

Lagoon Train:

- 1 Completely Mixed Aeration Cell with a total volume of 958,000 $ft^3$and coarse-bubble diffusers with 9,000 cfm per cell
- 5 Partially Mixed Aeration Cells each with a volume of 2,215,000 ft^3 and 5-11 floating aerators per cell

The installed dry-weather capacity of the Influent Pumping Station is approximately 31 mgd (24 mgd firm) with 58.5 mgd installed capacity in wet-weather pumps. When flow exceeds 24 mgd at the Influent Pump Station (or 34 mgd total at the flow control structure) the wet-weather pumps convey the excess flow to the head of the partially mixed aeration cells (Cell No. 3 and 4), bypassing the flow control structure, grit removal basins, and completely mixed Cell No. 1. The flow from the lagoon train is combined with the mechanical train effluent and flows by gravity through the effluent tunnel.

The gravity discharge effluent tunnel is a 96-inch HOBAS pipe tunnel that connects to the Kansas River effluent diffuser pipe. The existing diffuser was designed to discharge up to 105 mgd through the 24-inch check valves. The check valves can be upsized in the future for flows exceeding 105 mgd. Upsizing to 36-inch diameter check valves will increase the capacity of the diffuser to 132 mgd.

The watershed consists of a combination of mature suburban development, new suburban development, large commercial properties, business parks, and large plots of undeveloped land. Development began in the eastern portion of the watershed around the 1960's. The western portion of the watershed is mostly newer development starting around 1990. Previous reports have estimated that currently the Mill Creek Watershed is approximately 60% developed. It is estimated that development will continue throughout the watershed until the ultimate conditions are achieved.

1.3 PAST AND REFERENCE REPORTS

Mill Creek Watershed Alternatives Analysis and Optimization – HDR, 2017

The purpose of this report was to develop the long-term improvements plan for the collection system in the Mill Creek Watershed to ensure JCW's collection system level of service can be achieved and maintained for existing and future conditions. This included evaluating the need for conveyance system improvement alternatives to address existing capacity constraints, as well as future capacity issues due to growth within the watershed.

This report indicates the watershed is approximately 60% developed, and significant growth is expected to continue to occur. This growth is expected to increase flows in portions of the collection system to levels beyond what the existing infrastructure can convey. To address these capacity concerns a phased improvements plan was recommended to address the long-term growth in the watershed. The recommended phased improvements plan includes a combination of conveyance improvements, storage facilities, treatment plant improvements and/or infiltration and inflow (I/I) reduction in specific areas. Along with the alternative analysis, the automated optimization process was evaluated to determine the most beneficial application of the optimization software tools, in respect to both value added in the planning process and cost effectiveness.

Mill Creek Flow Monitoring, Modeling, and Planning – George Butler Associates, 2014

The purpose of this project was to evaluate the need for collection system capacity improvements for the Mill Creek Watershed, and more specifically capacity improvements for the Mill Creek Interceptor. A secondary goal was to determine areas or subsystems with excessive I/I within the watershed and identify subsystems for I/I improvement.

The capacity evaluations were performed for existing conditions, interim growth and ultimate growth. Flow monitoring was performed for both the 2012 and 2013 wet weather periods (April thru June). Flow monitoring data was used to calibrate a model for the existing network. Network models depicting the interim growth and ultimate growth scenarios were also developed. The InfoWorks models were used to identify bottlenecks in the system for the existing and future growth scenarios. Using the InfoWorks results, network improvement options were evaluated. The outcome of this report was the flow monitoring data and a phased improvement plan, which was the basis of the HDR report.

Nutrient Removal Pre-Design Study – Black & Veatch, 2010

The purpose of this report was to present the evaluation conducted to assess the feasibility of incorporating nutrient removal facilities and processes at Mill Creek Regional Wastewater WWTP. This evaluation was targeted at meeting 3 different levels of nutrient removal goals. Goal Level 1 was 8 mg/L Total Nitrogen (TN) and 1.5 mg/L Total Phosphorus (TP). Goal Level 2 was for 5 mg/L TN and 0.5 mg/L TP. Goal Level 3 was for 3 mg/L TN and 0.3 mg/L TP. The project team evaluated operational changes, biological treatment additions, and physical and chemical treatment additions needed to meet the levels. Opinions of probable operation and capital costs were developed, life cycle assessments including carbon footprint modeling were completed, and social-environmental impact analysis were performed to identify which alternative best meets the goals and objectives for this study.

2.0 Existing and Future Flows Analysis

2.1 EXISTING FLOWS

Existing dry weather flows from January 2014 through August 2019 were evaluated to develop an estimate of the current annual average (AA) and peak flow. Historical MCR influent flow data was provided by JCW and was used as the basis of analysis. Based on the historical average data since 2014, the average daily flow is 10.52 mgd. The peak day flow was recorded in May 2019 and was 72.72 mgd. Figure 2-1 presents the daily flow rates since 2014, a rolling 30-day average, and two trendlines of the historical daily average flow.

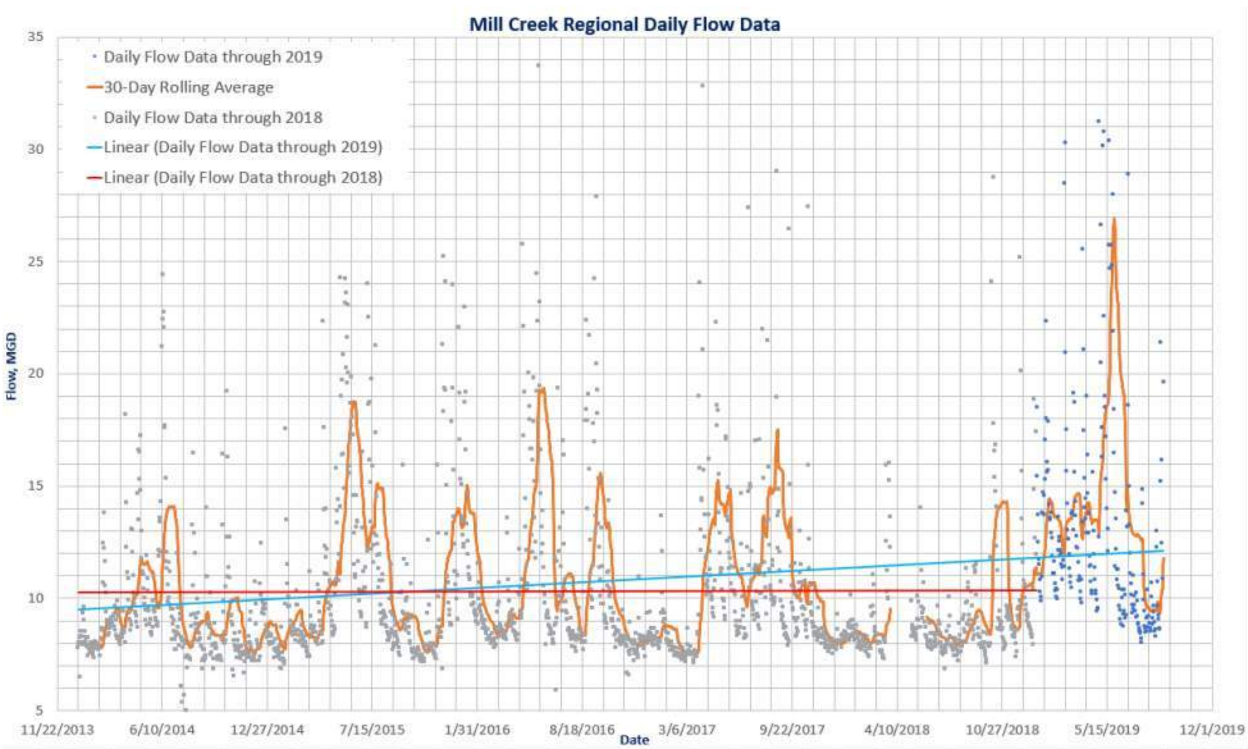

Figure 2-1 MCR Total Daily Flows (2014-2019)

Figure 2-1 shows two different trendlines. The trendline shown in blue is the daily average flow from 2014 through 2019. This trendline shows a slight increase in flow, but a very weak correlation of flow to time. It should be noted that the data for 2019 is not a full year of data. Typically, in the area served by MCR, the wetter part of the year occurs in the first two-thirds of the year. Precipitation in the last third of the year is generally less. It is possible that if there was a full year of data for 2019 the AA would be less than the current value. In addition, it should be noted that 2019 has been one of the wettest years in the history of the area. The red trendline shown on Figure 2-1 is for the average daily flow from 2014 to 2018. By excluding 2019 data, the slope of the trendline is essentially flat. Based on this trendline comparison, it can be assumed that the increase in flow for 2019 is correlated to the increase in rainfall.

One way to approximate the dry weather AA flow is to compare it to the annual rainfall volume for that given year. When annual rainfall is above average, the AA flow to the plant is increased primarily due to infiltration and inflow (I/I). When annual rainfall is below average the AA flow to the plant is decreased due to less than average I/I. To better estimate the dry weather AA flow to MCR a non-weighted average of 11 STORMWatch rain gauges located throughout the Mill Creek watershed area were analyzed. The national climate data for this area indicates an average annual rainfall of 38.86 inches. Table 2-1 shows a comparison of MCR AA flow versus annual rainfall totals for the Mill Creek watershed.

Table 2-1 MCR Annual Flow and Rainfall (2014 – 2019)

YEAR	ANNUAL RAINFALL TOTALS (IN)[1]	AA FLOW TO MCR (MGD)[2]
2014	32.7	9.4
2015	46.3	10.9
2016	41.5	10.9
2017	51.1	10.8
2018	33.4	9.5
2019[2]	41.2	14.3

[1] Annual rainfall totals, are the non-weighted averages of 11 STORMWatch Rain Gauges within the watershed.
[2] Flow data for 2019 is through August 31, 2019.

Table 2-1 shows a correlation between annual rainfall and MCR AA flow. Figure 2-2 graphically illustrates the information from Table 2-1. The correlation of annual rainfall to MCR AA flow is more pronounced in Figure 2-2.

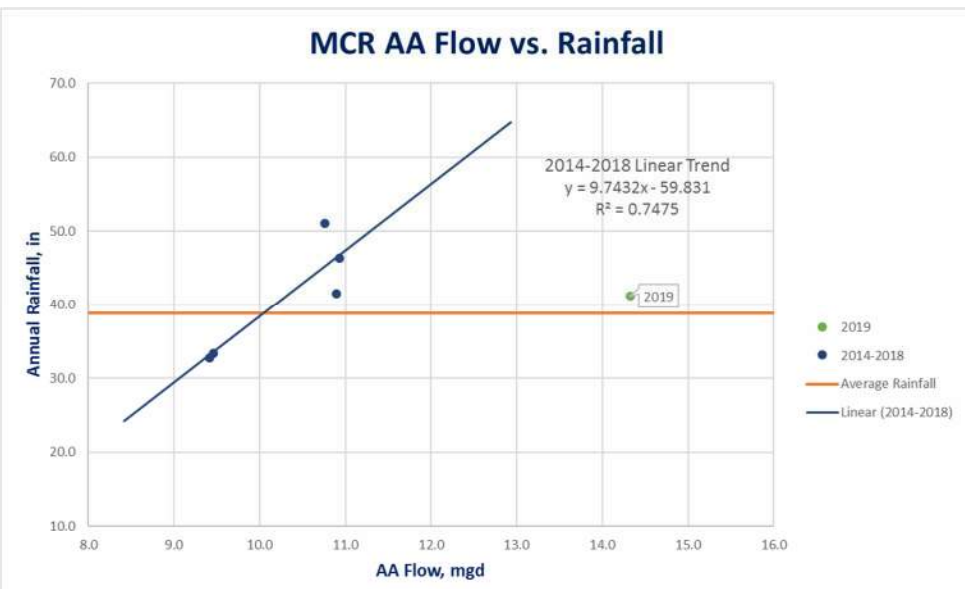

Figure 2-2 MCR Average Annual Flow vs. Rainfall (2014 –2019)

Figure 2-2 shows that the drier years correlate to a decreased AA flow, and wetter years correlate to an increase in AA flow. This is due to a decrease or increase in I/I for the given year. More rainfall results in more I/I. As previously noted, 2019 is somewhat of an outlier so it is not included when creating the trendline. From Figure 2-2, the MCR AA flow at average rainfall conditions is approximately 10.05 mgd. This is similar to the average of the historical daily flow of 10.52. Based on these two values it can be approximated that the current MCR AA flow is 10.5 mgd.

JCW's consultant for the watershed area collection system, HDR, developed and calibrated a hydraulic model of the Mill Creek watershed service area collection system. This hydraulic model was used to predict peak wet weather flows to MCR. HDR documented the comprehensive results of the collection system modeling in the Mill Creek Watershed Alternatives Analysis and Optimization Report, August 2017. This report expanded the modeling effort which was originally started by George Butler Associates' (GBA) Mill Creek Flow Monitoring, Modeling, and Planning Report, completed in 2014. Both of these reports are summarized in Section 1.3 of this TM.

The HDR Report indicates at the time of the hydraulic model development, the Mill Creek basin was approximately 60% built out. The report states that at the existing population, a 10-year storm event would produce an unrestricted flow (free flow conditions, with capacity restrictions removed) of 107 mgd.

2.2 FUTURE FLOWS

2.2.1 Dry Weather Flows

As previously stated, the HDR report estimates the Mill Creek watershed is approximately 60% developed. It is anticipated development will continue until ultimate conditions are reached. The HDR report also indicates that ultimate conditions equate to a population of 239,000. The current population is approximately 127,000. This increase of 112,000 people will have a significant impact on the AA and peak flow to MCR. JCW confirmed that these population numbers should be used for this facility plan. However, these estimates should be confirmed prior to beginning a future project impacted by these projections.

In the past, when looking at the MCR watershed continued growth was anticipated to an ultimate flow of 24 mgd. Because of this, the mechanical plant expansion completed in 2006, was sized to handle 24 mgd. In the last 20 years across the industry, average per capita usage has decreased in part due to low-flow plumbing fixtures and public awareness of water conservation. Using the 24 mgd ultimate flow with the HDR projected ultimate population results in approximately 100 gallons per day per capita (gpcd). In 2012, the GBA report used flow sampling to determine a gpcd of 78. This decrease in per capita usage confirms the industry trends are occurring across the watershed. For planning purposes, it is prudent to add a 10% increase to this 78 gpcd. This safety factor accounts for fluctuations in precipitation from year to year. To estimate the AA flow at an ultimate population of 239,000, multiply by the safety factor adjusted gpcd of 85.8 to equal 20.51 mgd. When designing a WWTP typically whole numbers are used as AA flow. Therefore, the 20.51 mgd can be rounded up for an ultimate AA flow of 21 mgd, or four trains of 5.25 mgd each.

To determine the design flow for this facility plan, an estimate of when the ultimate AA flow will occur is needed. Figure 2-3 depicts a range of possible growth rates and their impact on future flow rates over time.

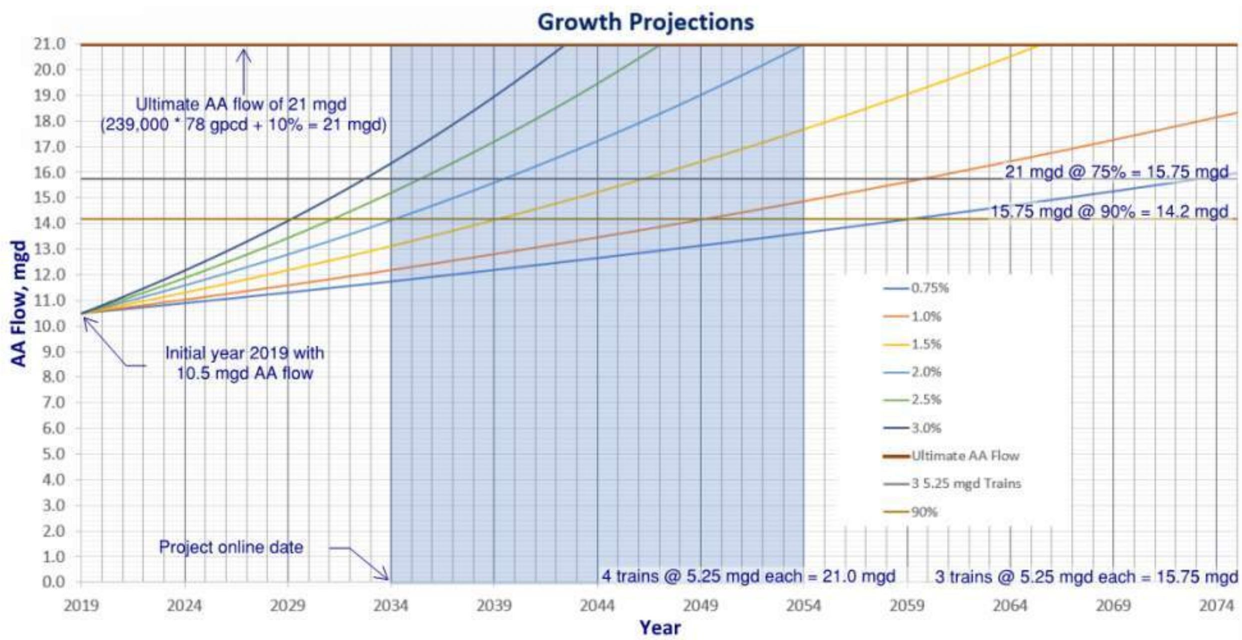

Figure 2-3 Mill Creek Regional Growth Projections

Figure 2-3 displays growth rates from 0.75% to 3%, shown in various colors. The previously described ultimate AA flow condition is shown on the top of the graph at 21 mgd. It is estimated that the nutrient improvements project will be online by 2034. Typically, large improvements projects of this sort need to have a long useful life of at least 20 years, which is indicated in the blue box. Lastly, there are horizontal lines at 14.2 mgd and 15.75 mgd. The 15.75 number is 75% of the ultimate AA flow of 21 mgd, or three of the total four process trains. Typically, at 90% plant capacity it is best practice to begin expansion activities if future growth is expected to continue. The 14.2 mgd flow is 90% of 15.75 mgd.

In the HDR Report, a 3% growth rate was used to be conservative. Typically, a 2-3% growth rate is assumed for planning purposes. If a 3% growth rate is used MCR will be at ultimate AA flow by roughly 2042, which means that the ultimate conditions should be the basis of design for the nutrient improvements project. If a lower growth rate of 1.5% is used ultimate conditions are not achieved until 2065, which may constitute designing for interim condition of 15.75 mgd and waiting on ultimate buildout in the future. However, 90% of that interim condition would be achieved approximately 5 years after construction of the project, which is not practical. Phased construction may be viable if the growth rate is less than 1%. As previously stated, this is a lower growth rate than is typical for planning projects. Therefore, it is recommended to use the ultimate AA flow of 21.0 mgd as the design AA flow for the MCR Facility Plan.

It has long been a consideration that the City of Olathe may decommission their Harold Street WWTP. This plant lies at the upper (southern) reach of the Mill Creek drainage area. The plant is permitted to 3.2 mgd AA with peak flow treatment up to 25 mgd. However, AA flow at the plant is 2 mgd with no increase expected. Given the existing JCW collection system, only dry weather flows would potentially be routed to MCR WWTP. The safety factor included in the ultimate design AA flow for MCR would accommodate the additional AA flow. Harold Street WWTP receives primarily municipal wastewater which should have a similar characterization to the influent reviewed for MCR and described in Section 3.

2.2.2 Wet Weather Flows

To approximate the future wet weather flows, the first step was a review of the collection system modeling reports by HDR and GBA. Both reports modeled peak flows for the ultimate peak flow condition. The HDR Report indicates that 146 mgd is the 10-year design storm unrestricted peak flow to MCR for the ultimate population condition. However, in that HDR Report there are several recommended phased improvements to the collection system including IPS upgrades, I/I reduction, underground storage facilities, and interceptor capacity improvements. HDR has indicated that at the completion of all the collection system improvements, the peak flow to the WWTP will be 116 mgd. If 116 mgd is the ultimate peak flow and 21 mgd is the ultimate AA flow that means the peaking factor is 5.52. It is recommended that this peaking factor be rounded up to 6.0, to provide a margin of safety, giving a peak wet weather ultimate flow to MCR of 126 mgd. As previously mentioned, if the 24-inch check valves in the gravity effluent tunnel get switched out to 36-inch check valves, the diffuser capacity is 132 mgd. This means at the ultimate growth conditions the effluent tunnel will still be sized appropriately to convey flow to the Kansas River.

To validate the ultimate wet weather peak flow to MCR, along with the maximum month (MM) peaking factor historical MCR data was analyzed and compared to other JCW WWTPs. This comparison is shown in Table 2-2.

Table 2-2 MCR Maximum Month Average Day and Peak Flows (2014 –2019)

YEAR	AA (MGD)	MM:AA	PD:AA	MAX:AA
MCR Nutrient Pre-Design Study	24.0	1.47	-	-
Tomahawk Creek Design	19.0	1.39	5.55[1]	9.05
Blue River Main Design	7.0	1.46	-	6.00
2014 MCR Flow Data	9.4	1.50	3.15	-
2015 MCR Flow Data	10.9	1.51	3.51	-
2016 MCR Flow Data	10.9	1.76	5.59	-
2017 MCR Flow Data	10.8	1.45	6.12	-
2018 MCR Flow Data	9.5	1.49	5.50	-
2019 MCR Flow Data[2]	14.3	1.81	5.08	-
MCR Design Flows	21.0	1.50 (31.5 mgd)	-	6.00 (126.00 mgd)

[1] Based on sewershed modeling peak day flow of 105.4 mgd
[2] Data is through August 31, 2019

Looking at Table 2-2, MM and MAX peaking factors of 1.50 and 6.00 are similar to the historical range at MCR and other JCW WWTPs. The ratio of peak day (PD) to AA is something that can be calculated given a range of flows, but it is not something that is typically designed for full biological treatment. The important thing to note about the PD:AA column of Table 2-2 is that the peaking factor is generally below 6.0, which is the recommended basis of design peak flow.

2.2.3 Recommended Design Flows

Since there is still significant development to occur in the Mill Creek Watershed it is important to understand the design flows across all possible flows to the plant to ensure equipment turndown requirements. It can be assumed that AA Startup conditions will be less than the AA Ultimate condition because even at an aggressive growth rate of 3 percent, the watershed is not completely built out by 2034. It can also be assumed that the AA Startup condition will have a diurnal low AA flow associated with it based on industry trends. A conservative approach to estimating AA Startup conditions is to use a 1 percent population growth rate from Figure 2-3. Historically for this area, a diurnal low condition can be approximated to be roughly half of the diurnal high AA condition. A summary of the recommended design flows are shown in Table 2-3. Based on this Table an equipment turndown factor between MM and diurnal low AA startup can be approximated at 5.25:1.

Table 2-3 MCR Recommended Design Flows

	DIURNAL LOW AA STARTUP	AA STARTUP	AA ULTIMATE	MAX MONTH	PEAK DAY
MCR Design Flows	6.00	12.00	21.0	31.5	126.0

1 Historically this is 1/2 of the diurnal high (AA startup)
2 Flow projection based on Figure 2-3, year 2034 startup, assuming 1% growth

3.0 Existing and Projected Future Loads

MCR daily average influent data, ranging from September 5, 2014 to August 31, 2019, was evaluated to develop design temperature, design pH, and the annual average (AA), maximum monthly average (MM), and peak day (PD) mass influent concentrations and loads.

3.1 APPROACH

The following approach was taken to select concentrations and loads for design:

1. *Review data quality.* Erroneous data due to data entry mistakes and malfunctioning equipment were removed from the data set with input from JCW. Influent composite samples were typically taken four times per month, though BOD and TSS were sampled on a more frequent basis between September 2016 to December 2016. Load data was unavailable in April 2018 due to a malfunctioning flowmeter.

2. *Evaluate trends in influent parameter concentrations over time*. It has been a common observation at utilities across the nation that domestic wastewater concentrations are increasing while flows remain the same or increase at a lower rate. National trends show increasing use of water conserving fixtures, which reduces per capita water use but does not impact per capita mass contributions. Historical trends in the influent parameter concentrations were evaluated through linear regression of the dataset over time and an assessment of AA values over 5 years.

3. *Calculate loads and peaking factors.* Daily average mass loads were calculated for BOD, TSS, TKN, and TP. From these values, AA, MM, and PD load values were determined on an annual basis for comparison and calculation of the MM:AA and PD:AA peaking factors.

4. *Select the AA load used to determine design concentrations*. The AA load value used to determine design concentrations was an average of the 5 year dataset from 5 September 2014 to 5 September 2019.

5. *Select MM:AA and PD:AA peaking factors used to determine design conditions*. Selection of peaking factors used to determine design conditions was based on the historical MCR annual peaking factors and compared to values used in the 2010 Nutrient Removal Pre-Design Study, the THC WWTF Design, and the BRM WWTF Phase I Improvements. The peaking factors were used to calculate MM and PD loads from the AA value determined in Step 4.

6. *Calculate AA and MM concentrations using the AA and MM loads from Steps 4 and 5 and flows from Section 2.*

7. *Calculate influent parameter ratios and their probability.* The parameter ratios of VSS:TSS, NH_4-N:TKN, and OP:TP were calculated on a daily average basis for the 5 year dataset. The 50^{th} percentile values were used to calculate VSS, NH_4-N, and OP concentrations and loads, given the TSS, TKN, and TP concentrations calculated in Step 6.

8. *Calculate the design loads*. The concentrations calculated in Step 7 using historical flows and loads were applied to the design year without adjustment. The design flows determined in Section 2 were used to calculate design loads.

3.2 TEMPERATURE ANALYSIS

Monthly average temperatures are plotted for January 2015 through August 2019 in Figure 3-1. The monthly averages are comprised of 3-5 measurements per month. From the monthly averages, the minimum, average, and maximum temperatures were selected for design as shown in Table 3-1. Notably, the MCR minimum monthly average temperature differs from the THC WWTF design, but the monthly average temperature of 10.5 °C or below was observed on multiple occasions in January and February (i.e., 3 months total).

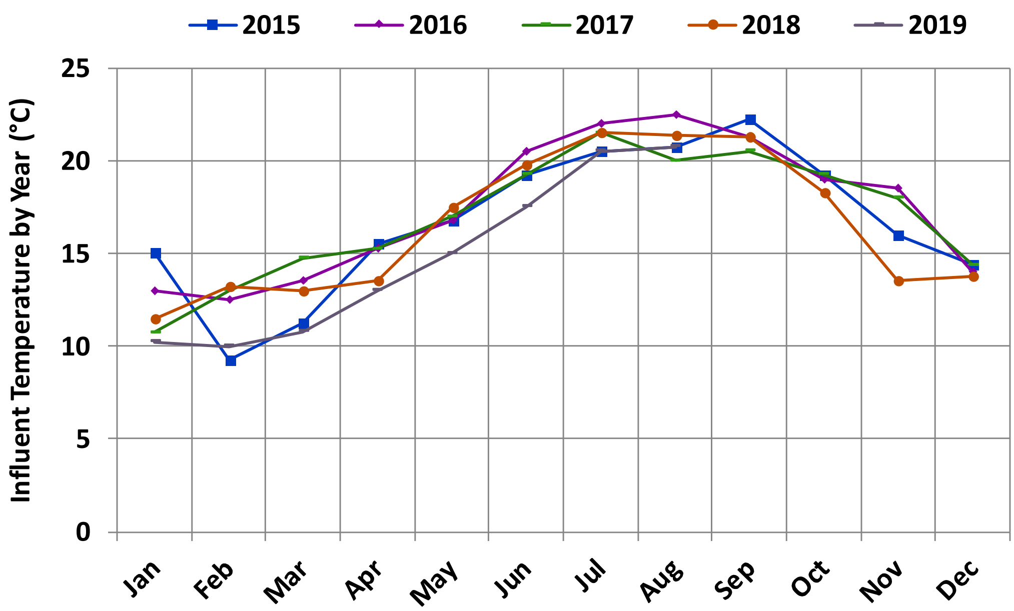

Figure 3-1 MCR Influent Temperature Monthly Averages 2015-2019

Table 3-1 MCR Temperature Summary

SCENARIO	MCR DATASET(°C)	THC DESIGN(°C)	SELECTED FOR MCR DESIGN (°C)
Min. (Winter)	9.3	13	10
Average	16.6	18	16.5
Max. (Summer)	22.5	23	23

3.3 PH AND ALKALINITY ANALYSIS

The influent pH data is plotted in Figure 3-2. No alkalinity data was available in the evaluated dataset, however the MCR mechanical plant currently fully nitrifies with partial denitrification and has no issues with alkalinity. It is assumed there will not be alkalinity/pH issues with the future plant. An average influent pH of 7.4 will be used in the process model.

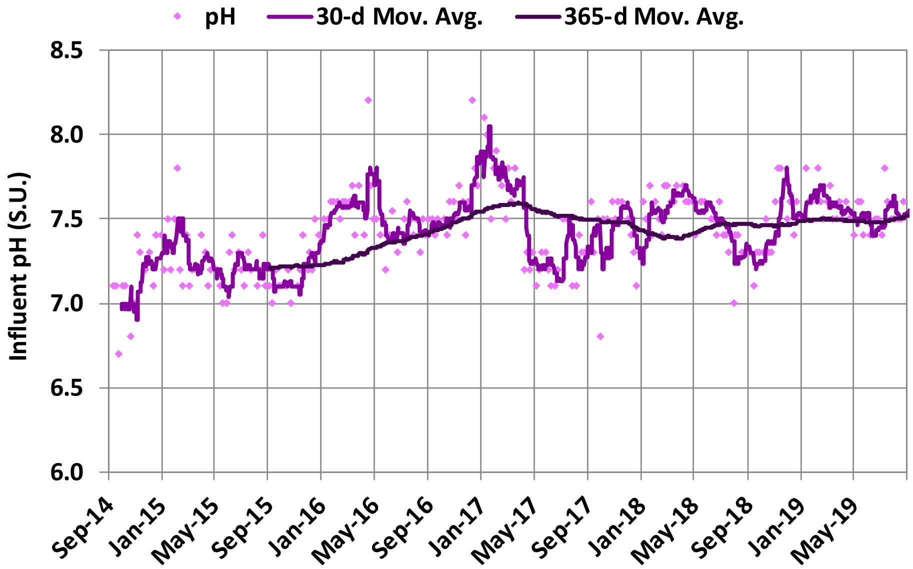

Figure 3-2 **MCR Influent pH 2015-2019**

3.4 BOD LOAD ANALYSIS

The linear trend in the daily average concentration data shown in Figure 3-3 experienced a nearly flat slope, meaning the BOD concentrations did not change significantly over time according to the linear regression. The annual average values in Figure 3-4 also show no clear positive or negative trend. The lowest concentration was experienced in 2019, while the highest concentration was experienced in 2018. Of note, 2018 experienced the lowest AA flow, while 2019 experienced the highest AA flow and extreme wet weather. This same observation is made for all the constituents, including TSS, TKN, and TP. Also note that 2014 is omitted from the figure due to the dataset only including 4 months of 2014 data. The year 2019 is shown, though it is an average of January - August data only.

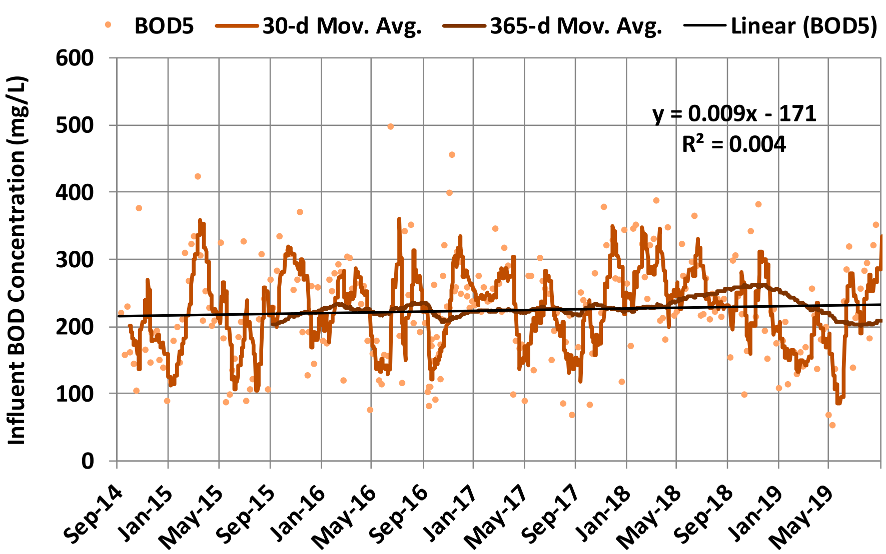

Figure 3-3 **MCR Daily Average Influent BOD Concentrations 2015-2019**

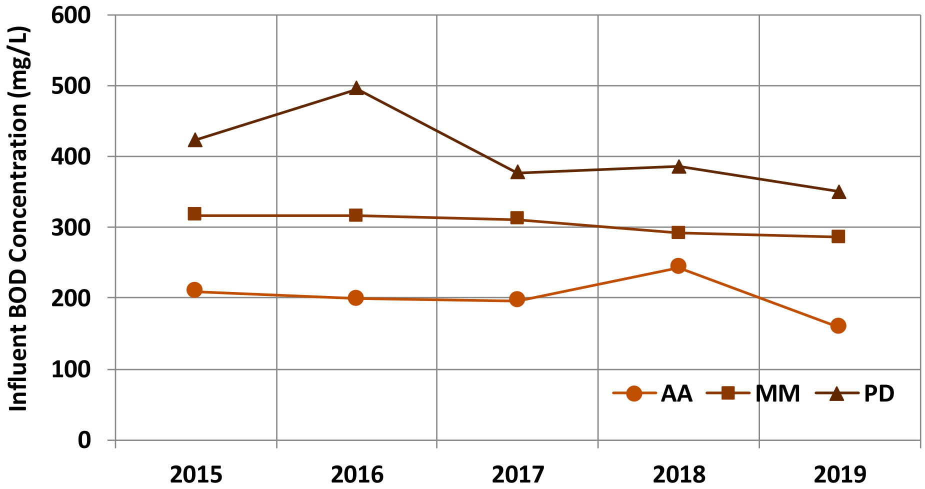

Figure 3-4 MCR Annual Average, Maximum Month, and Peak Day Influent BOD Concentrations 2015-2019

Table 3-2 provides the AA, MM, and PD BOD loads, and corresponding peaking factors. The maximum observed MM:AA peaking factor was 1.26. This number was rounded up to 1.30 for the selected MM:AA peaking factor carried forward for calculation of MM loads. A MM:AA peaking factor of 1.30 is equal to the value used for the THC WWTF design. Note that MCR monthly averages were typically based on only 4 data points, which makes it challenging to assess the monthly average with precision. The maximum observed PD:AA peaking factor was 2.07, with the second greatest value being 1.60. Although there is a large gap between the first and second values, the three reference design values all exceed the second highest value so 2.0 was selected as the PD:AA peaking factor.

The higher MM:AA peaking factor is a bit conservative but practical for this study. It is not known how the wastewater characteristics may change over the coming years. A higher MM:AA BOD design load will increase the size of the secondary process.

Table 3-2 Historical BOD Load Summary

	ANNUAL AVERAGE (PPD)	ANNUAL MAX MONTH		ANNUAL PEAK DAY	
		Load (PPD)	Peaking Factor	Load (PPD)	Peaking Factor
MCR Nutrient Pre-Design	--	--	1.18	--	1.84
THC WWTF Design	--	--	1.30	--	1.81
BRM WWTF Design	--	--	1.57	--	1.98
MCR 2014 Data[1]	12,220 (n=15)	--	--	--	--
MCR 2015 Data	19,100 (n=48)	24,130 December (n=4)	1.26	30,550 03-Dec	1.60
MCR 2016 Data	18,170 (n=67)	22,080 November (n=6)	1.22	37,630 16-Jun	2.07
MCR 2017 Data	17,670 (n=47)	21,480 December (n=4)	1.22	26,550 06-April	1.50
MCR 2018 Data	19,130 (n=43)	22,930 March (n=4)	1.20	27,710 15-Nov	1.45
MCR 2019 Data[2]	18,910 (n=30)	21,700 August (n=4)	1.15	26,450 22-Aug	1.40
Selected MCR Design	**18,100**		**1.30**		**2.00**

[1] 2014 is based on data from September – December only. Peaking factors were not developed for this partial year, which is less than half of a year.

[2] 2019 is based on data from January-August only

3.5 TSS AND VSS LOAD ANALYSIS

Overall, the linear regression trend in Figure 3-5 indicates a 1% growth in TSS concentration per year for the complete dataset. However, the AA concentrations depicted in Figure 3-6 show no clear trend. Like BOD, the highest AA concentration occurred in 2018, which experienced the lowest flows, and the lowest AA concentration occurred in 2019, which experienced the highest flows and extreme wet weather. Also note that 2019 average is based on an incomplete year.

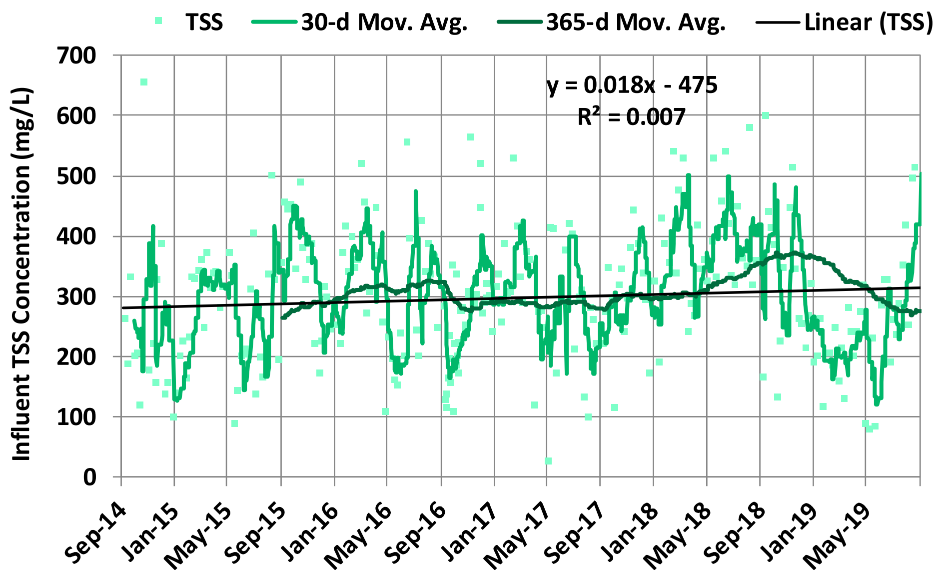

Figure 3-5 **MCR Daily Average Influent TSS Concentrations 2015-2019**

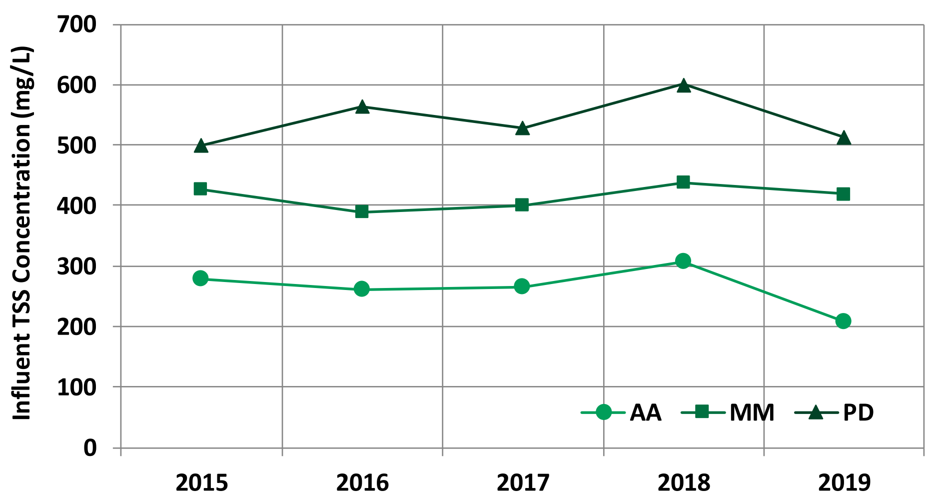

Figure 3-6 **MCR Annual Average, Maximum Month, and Peak Day Influent TSS Concentrations 2015-2019**

Table 3-3 provides the AA, MM, and PD TSS loads and corresponding peaking factors. A MM:AA peaking factor of 1.30 was selected considering the highest observed MM:AA peaking factors were 1.29 and 1.28. It was decided to not use the MM:AA peaking factors of the THC and BRM designs (i.e., 1.42, 1.49) as the values were significantly greater than those observed at MCR.

The maximum observed PD:AA peaking factor was 2.25, which occurred in 2017. However, 2.25 greatly exceeds the second highest value of 1.89. It was decided to use a PD:AA peaking factor of 2.0, which matches with the THC WWTF design value and falls between 1.89 and 2.25.

The higher MM:AA peaking factor is conservative, but practical for this study. It is unknown how the wastewater characteristics may change over the coming years. A higher MM:AA BOD design load will increase the size of the secondary treatment process. MCR receives hauled waste which is included in the influent sample. The MM:AA values from the site data average 1.23. A 1.3 MM:AA ratio is a typical ratio for many medium size watersheds and matches with the MM:AA ratio chosen for BOD.

Table 3-3 Historical TSS Load Summary

	ANNUAL AVERAGE (PPD)[1,2]	ANNUAL MAX. MONTH		ANNUAL PEAK DAY	
		Load (PPD)	Peaking Factor	Load (PPD)	Peaking Factor
MCR Nutrient Pre-Design	--	--	1.27	--	--
THC WWTF Design	--	--	1.42	--	2.02
BRM WWTF Design	--	--	1.49	--	2.22
MCR 2014 Data[1]	16,530 (n=15)	--	--	--	--
MCR 2015 Data	25,380 (n=48)	32,660 December (n=4)	1.29	48,090 17-Dec	1.89
MCR 2016 Data	23,770 (n=67)	29,030 June (n=6)	1.22	42,180 16-Jun	1.77
MCR 2017 Data	23,900 (n=47)	29,710 August (n=4)	1.24	53,700 24-Aug	2.25
MCR 2018 Data	27,750 (n=42)	31,520 June (n=4)	1.14	41,950 13-Sep	1.51
MCR 2019 Data[2]	24,830 (n=32)	31,700 August (n=4)	1.28	38,770 22-Aug	1.56
Selected MCR Design	**24,500**		**1.30**		**2.00**

[1] 2014 is based on data from September – December only. Peaking factors were not developed for this partial year, which is less than half of a year.

[2] 2019 is based on data from January-August only

The influent VSS:TSS ratio daily averages are plotted against time in Figure 3-7. The 30-day running average generally fell within the range of 0.8 to 1.0. The probability plot is provided in Figure 3-8. The 50th percentile value of 0.88 was selected for the projection of VSS loads. For comparison, the value used in the THC WWTF design was 0.90. Both values indicate a fresh wastewater with little hydrolysis and fermentation occurring in the collection system.

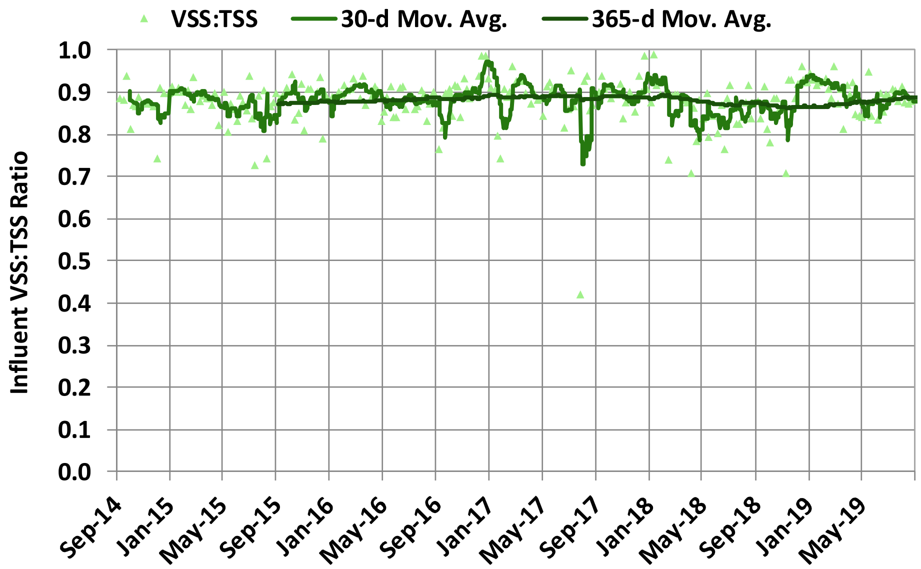

Figure 3-7 MCR Daily Average Influent VSS:TSS Ratio 2015-2019

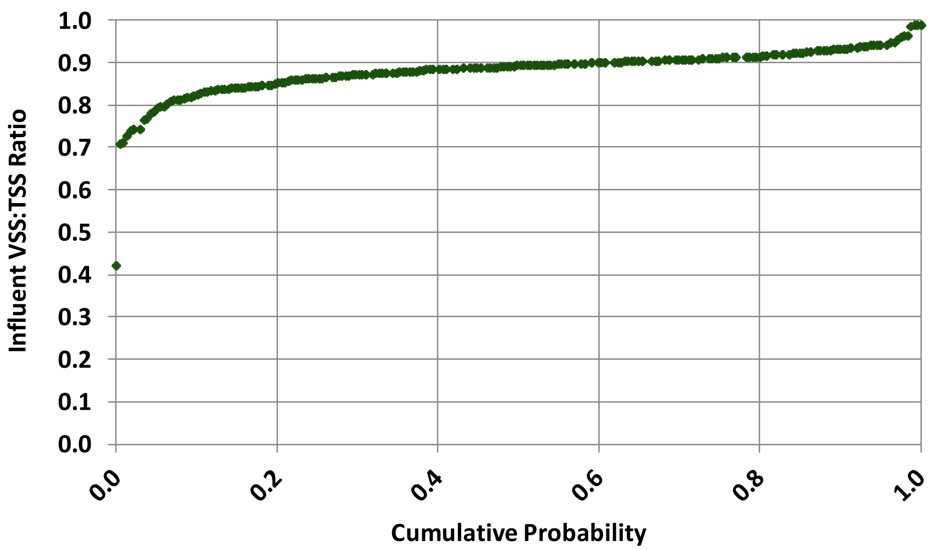

Figure 3-8 MCR Daily Average Influent VSS:TSS Ratio Probability 2015-2019

3.6 TKN AND AMMONIA LOAD ANALYSIS

Overall, the linear regression slope shown in Figure 3-9 indicates a 0.5% decline in TKN concentration per year for the TKN dataset. However, the AA concentrations plotted in Figure 3-10 show no clear trend as the concentration oscillates up and down from year to year.

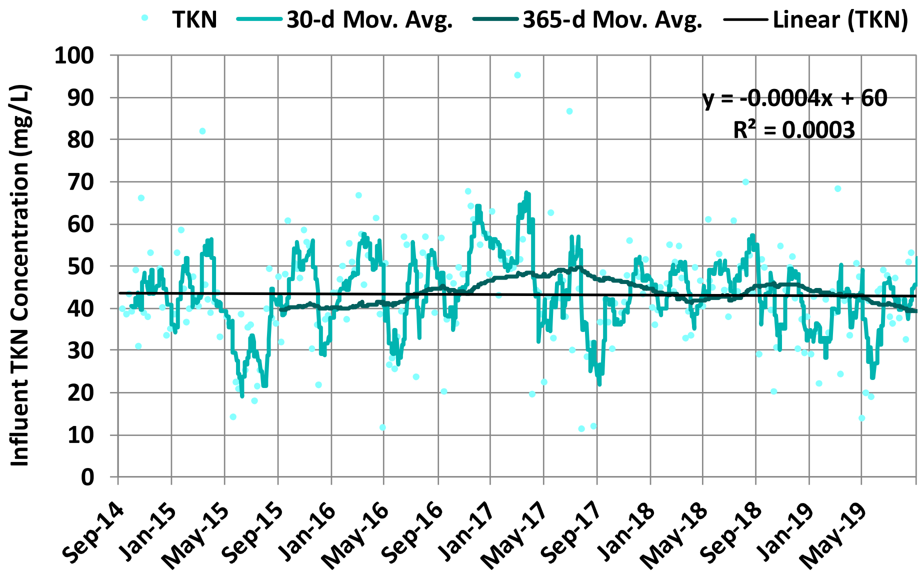

Figure 3-9 **MCR Daily Average Influent TKN Concentrations 2015-2019**

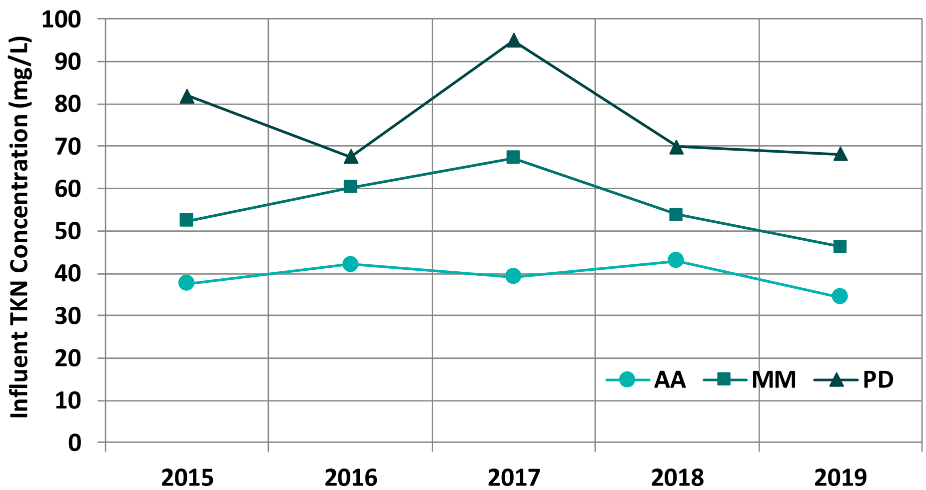

Figure 3-10 **MCR Annual Average, Maximum Month, and Peak Day Influent TKN Concentrations 2015-2019**

Table 3-4 provides the AA, MM, and PD TKN loads, and corresponding peaking factors. The maximum observed MM:AA peaking factor of 1.36 occurred in 2019, however 2019 values were based on an incomplete year. The next highest MM:AA peaking factor was 1.24, which occurred in 2015. A value of 1.27 was selected as the design value, which is lower than the 2019 value, but slightly higher than the 2015 value, and is the average of the MCR Nutrient Pre-Design, THC, and BRM design values. The maximum PD:AA peaking factor was selected as 1.75. This value is greater than the THC and BRM WWTF design values, however, with a PD:AA peaking factor of 1.75 experienced in 2015 and 1.72 experienced in 2017, the reoccurring value was deemed valid.

Table 3-4 Historical TKN Load Summary

	ANNUAL AVERAGE (PPD)	ANNUAL MAX. MONTH		ANNUAL PEAK DAY	
		Load (PPD)	Peaking Factor	Load (PPD)	Peaking Factor
MCR Nutrient Pre-Design	--	--	1.22	--	2.30
THC WWTF Design	--	--	1.23	--	1.49
BRM WWTF Design	--	--	1.36	--	1.55
MCR 2014 Data[1]	2,920 (n=15)	--	--	--	--
MCR 2015 Data	3,420 (n=47)	4,250 December (n=4)	1.24	5,990 17-Dec	1.75
MCR 2016 Data	3,840 (n=48)	4,540 May (n=6)	1.18	5,110 05-May	1.33
MCR 2017 Data	3,520 (n=48)	4,220 March (n=4)	1.20	6,030 29-Jun	1.72
MCR 2018 Data	3,380 (n=43)	3,830 August (n=4)	1.13	4,860 09-Aug	1.44
MCR 2019 Data[2]	4,100 (n=31)	5,590 May (n=4)	1.36	6,400 23-May	1.56
Selected MCR Design	**3,580**		**1.27**		**1.75**

[1] 2014 is based on data from September – December only. Peaking factors were not developed for this partial year, which is less than half of a year.
[2] 2019 is based on data from January-August only

The influent NH_4-N:TKN ratio is shown in Figure 4-11. No trend is discernable. The 50th percentile value of 0.54, shown in Figure 4-12, was selected for the calculation of ammonia loads. For comparison, the value used for the THC WWTF design was 0.51. Both values indicate a fresh wastewater with little hydrolysis and fermentation occurring in the collection system.

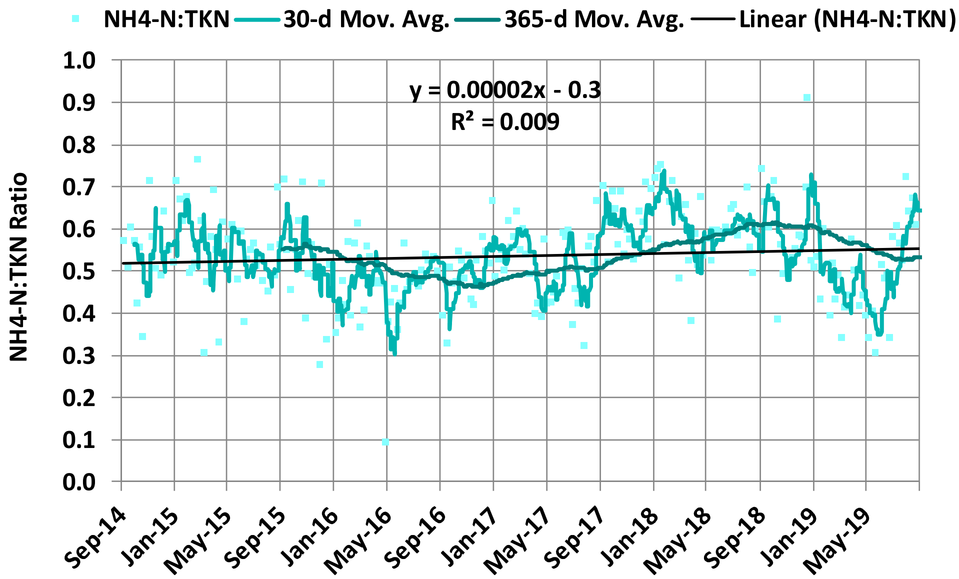

Figure 3-11 **MCR Daily Average Influent Ammonia:TKN Ratio 2015-2019**

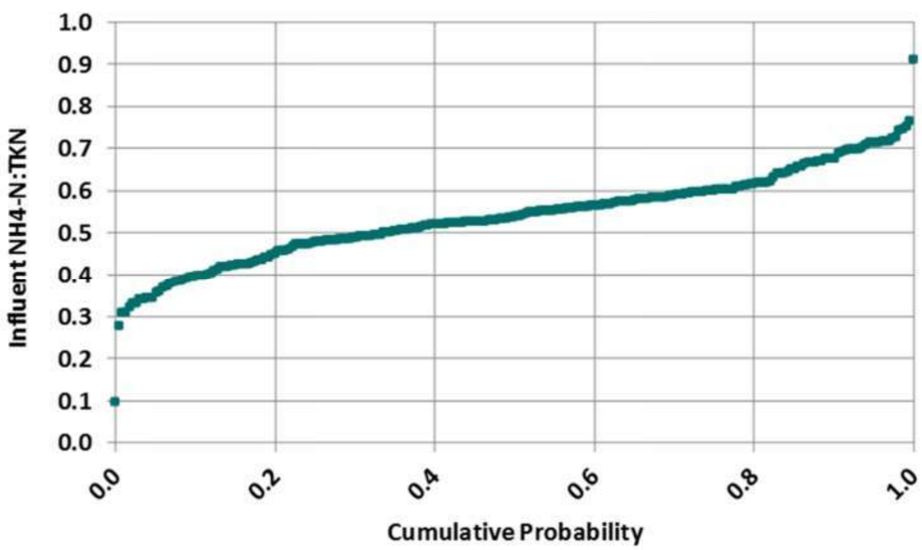

Figure 3-12 **MCR Daily Average Influent Ammonia: TKN Ratio Probability 2015-2019**

3.7 TP AND OP LOAD ANALYSIS

Overall, the linear trend in Figure 3-13 indicates a 2% growth in TP concentration per year for the entire dataset. However, the AA concentrations depicted in Figure 3-14 show no clear trend. Like other constituents, the highest AA concentration occurred in 2018, which experienced the lowest flows and the lowest AA concentration occurred in 2019, which experienced the highest flows.

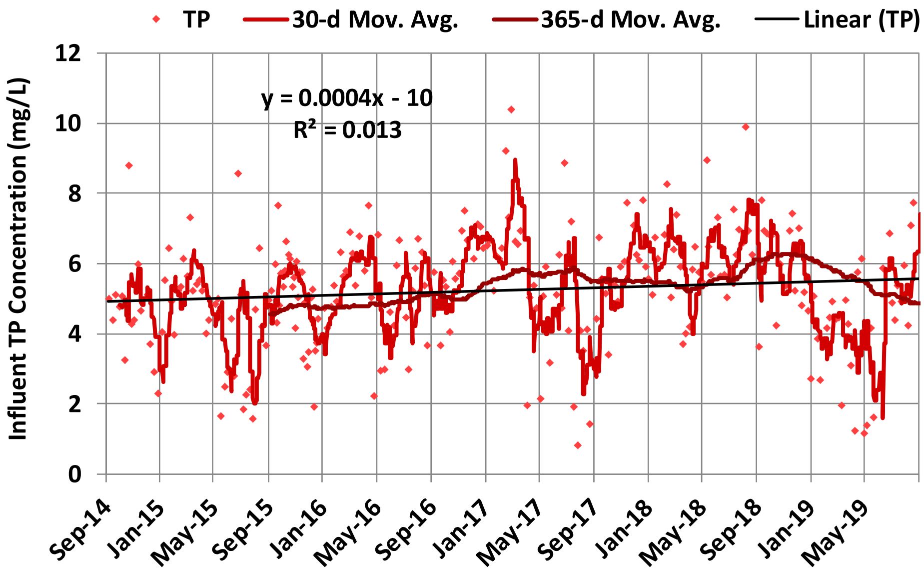

Figure 3-13 MCR Influent TP Concentration 2015-2019

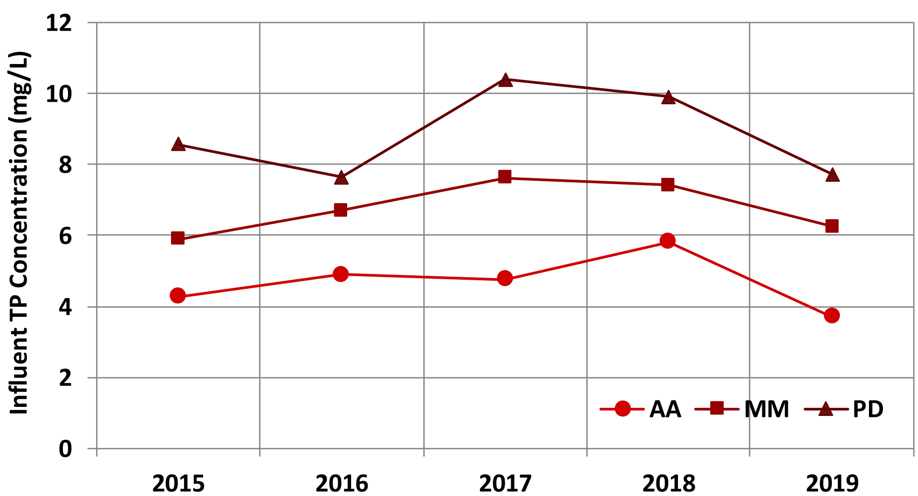

Figure 3-14 MCR Annual Average, Maximum Month, and Peak Day Influent TP Concentrations 2015-2019

Table 3-5 provides the AA, MM, and PD TP loads, and corresponding peaking factors. The MM:AA peaking factor of 1.30 was selected considering the 2016 and 2019 peaking factors of 1.31 and 1.35. The highest PD:AA peaking factor of 1.94 was experienced in 2019, but occurred in a year without a full dataset. Further, 1.94 was significantly higher than the other four annual peaking factors, which ranged from 1.50 to 1.68. A PD:AA peaking factor of 1.75 was selected, which fell between the THC and BRM values.

Table 3-5 Historical TP Load Summary

	ANNUAL AVERAGE (PPD)	ANNUAL MAX. MONTH		ANNUAL PEAK DAY	
		Load (PPD)	Peaking Factor	Load (PPD)	Peaking Factor
MCR Nutrient Pre-Design	--	--	1.55	--	--
THC WWTF Design	--	--	1.26	--	1.67
BRM WWTF	--	--	1.69	--	1.92
MCR 2014 Data[1]	320 (n=15)	--	--	--	--
MCR 2015 Data	390 (n=63)	480 June (n=4)	1.23	650 25-Jun	1.68
MCR 2016 Data	450 (n=47)	580 May (n=4)	1.31	740 26-May	1.66
MCR 2017 Data	430 (n=48)	480 March (n=4)	1.12	640 2-Mar	1.50
MCR 2018 Data	460 (n=47)	530 August (n=4)	1.15	730 6-Dec	1.59
MCR 2019 Data[2]	440 (n=31)	600 June (n=4)	1.35	860 27-Jun	1.94
Selected MCR Design	**420**		**1.30**		**1.75**

[1] 2014 is based on data from September – December only. Peaking factors were not developed for this partial year, which is less than half of a year.

[2] 2019 is based on data from January-August only

The influent OP:TP ratio is shown in Figure 3-15. The ratio was unusually high at the end of 2014 and the beginning of 2015. For subsequent dates, the ratio remained in a relatively tight range. The OP:TP probability is shown in Figure 3-16. The 50th percentile value of 0.41 was selected as the design value.

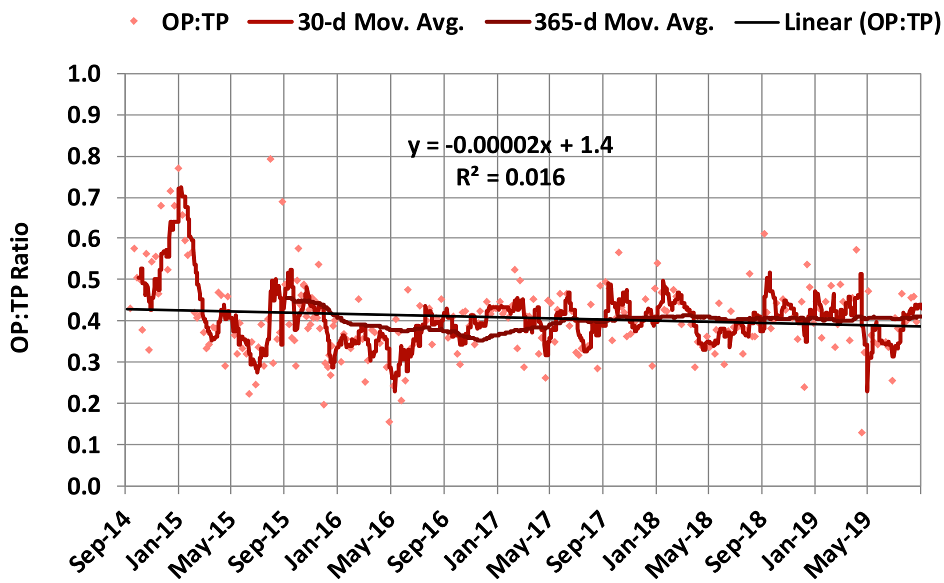

Figure 3-15 MCR OP:TP Ratio 2015-2019

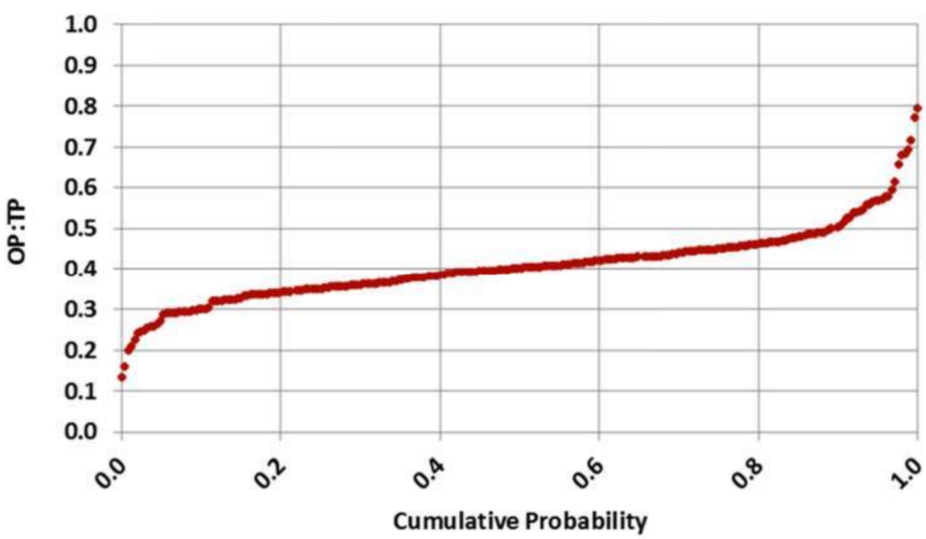

Figure 3-16 MCR OP:TP Ratio Cumulative Probability 2015-2019

3.8 Current Influent Concentrations and Comparison to Other JCW Facilities Data

Given the selected AA loads and PFs and the flows determined in Section 2, the following flows, loads, and concentrations presented in Table 3-6 were established.

Table 3-6 Current MCR Influent Flows, Loads, and Concentrations

PARAMETER	ANNUAL AVERAGE (AA)		MM:AA	MAXIMUM MONTHLY AVERAGE (MM)		PD:AA	PEAK DAY
Flow, mgd	10.5		1.50	15.8		--	--
	mg/L	ppd		mg/L	ppd		ppd
BOD	207	18,100	1.30	179	23,530	2.00	36,200
TSS	280	24,500	1.30	240	31,900	2.00	49,000
VSS	247	21,600	1.30	211	28,100	2.00	43,100
TKN	41	3,580	1.27	35	4,550	1.75	6,270
Ammonia	22	1,930	1.27	19	2,460	1.75	3,390
Total Phosphorus	4.8	420	1.30	4.2	550	1.75	740
Ortho-Phosphate	1.9	170	1.30	1.8	230	1.75	300

In preparation for the MCR design, it is recommended influent sampling occurs more frequently in the 5 years preceding design efforts. An increase in daily composite sampling frequency to 2-3 samples per week is suggested. In addition, a special sampling campaign should be conducted to capture the fractions of the carbonaceous, nitrogenous, and phosphorus species (i.e., particulate, colloidal or soluble, and biodegradable or nonbiodegradable fractions). Ideally, this special sampling will occur over multiple seasons and multiple years. One approach may be to sample additional analytes required for influent fractionation 2 times per month for the 5 years leading up to design. Alternatively, the influent fractionation may be conducted in a more intensive special sampling campaign over a shorter period (i.e., multiple seasons in a single year).

Bioxide (calcium nitrate) is currently used in the collection system to avoid anaerobic conditions in the pipe and mitigate odors. Consequently, the influent tends to have a higher particulate fraction, which negatively affects the availability of carbon for biological phosphorus removal. Particulate carbon is also more difficult to hydrolyze and will impact the anoxic zone denitrification efficiency. Ultimately, the use of Bioxide may increase the dose of supplementary carbon required to achieve the effluent nutrient limits.

Ferric chloride may be used in the collection system for odor control as a replacement for Bioxide. Ferric chloride would mitigate odors without stopping fermentation in the collection system and/or facilitating carbon use in the collection system via denitrification. Knowledge of the current MCR influent fractions would help to determine the need for this switch. It is also prudent to conduct a special sampling campaign once the chemical type and dosing strategy in the collection system are finalized, in order to have this information for design.

To validate the daily influent concentrations at MCR, they were compared to the influent quality at the BRM and THC WWTFs. These facilities were selected as treatment facilities that serve a similar socio-economic watershed, with no significant industrial component. Table 3-7 summarizes the influent pollutant concentrations at the three treatment facilities.

Overall, the MCR BOD and TSS concentrations are higher than those experienced at THC and BRM. For both MCR and THC, the MM concentrations are lower than AA concentrations due to a high flow peaking factor relative to load peaking factor.

MCR and BRM both experience higher TSS concentrations than BOD concentrations, which is typical for other JCW facilities. This observation indicates higher fractions of nonbiodegradable organic matter occur at MCR and BRM than at THC, which may be caused by the septage received at MCR and BRM. The MCR TKN concentration was higher than BRM, but lower than THC. Finally, the TP value was similar among all three facilities, with the exception of the BRM MM average value of 5.1 mg/L.

Table 3-7 Comparison of the MCR Flows and Loads to THC WWTF and BRM WWTF

	MILL CREEK REGIONAL WWTP		TOMAHAWK CREEK WWTF		BLUE RIVER MAIN WWTF		TYPICAL DOMESTIC WASTEWATER
	Annual Average	Maximum Monthly Average	Annual Average	Maximum Monthly Average	Annual Average	Maximum Monthly Average	Median Values
Flow, mgd	10.5	15.8	13.8	20.8	5.0	7.5	--
BOD, mg/L	206	178	154	141	181	192	200
TSS, mg/L	276	240	155	148	235	243	200
Ammonia, mg/L	22	18	29	24	21	19	25
TKN, mg/L	40	34	47	42	38	34	40
TP, mg/L	4.8	4.2	4.9	4.1	4.6	5.1	5

3.9 DEVELOPMENT OF FUTURE FLOWS AND LOADS

The flows developed in Section 2 and the influent wastewater concentrations from Table 3-6 were used to develop design influent concentrations and loads at MCR WWTF. Table 3-8 presents the recommended design flows and loads for use in the MCR Facility Plan.

Table 3-8 Recommended Basis of Design

PARAMETER	ANNUAL AVERAGE (AA)		MM:AA	MAXIMUM MONTHLY AVERAGE (MM)		PD:AA	PEAK DAY
Flow, mgd	21.0		1.50	31.5		6.0	126.0
	mg/L	ppd		mg/L	ppd		ppd
BOD	207	36,200	1.30	179	47,100	2.00	72,400
TSS	280	49,000	1.30	240	63,800	2.00	98,000
VSS	247	43,200	1.30	211	56,200	2.00	86,200
TKN	41	7,160	1.27	35	9,100	1.75	12,500
Ammonia	22	3,860	1.27	19	4,920	1.75	6,780
Total Phosphorus	4.8	840	1.30	4.2	1,100	1.75	1,480
Ortho-Phosphate	1.9	340	1.30	1.8	460	1.75	600

4.0 NPDES Limits

4.1 BACKGROUND

The existing National Pollutant Discharge Elimination System (NPDES) Water Pollution Control permit for MCR WWTP was issued by KDHE. The existing permit was issued for a design flow of 18.75 mgd and a peak wet weather flow of 34.0 mgd. The existing permit includes technology based effluent limits for BOD, TSS, pH, and water quality-based limits for ammonia, E. coli, and Whole Effluent Toxicity (WET) testing, and DO. Flow, total phosphorus, total nitrogen, and lead are monitored. A summary of existing permit limits is included in Table 4-1.

The permit requires MCR to maximize the flow through the mechanical plant up to its design capacity while diverting influent flow to the lagoons as necessary to maintain biological activity in the lagoon treatment system to achieve effluent permit requirements. The facility receives domestic wastewater from residential and commercial areas and a small fraction of industrial wastewater from local manufacturers.

4.2 FUTURE PERMIT LIMITS

The updated permit issued February 2020 is based upon an average discharge flow of 18.75 mgd to the Kansas River. The effluent limits are technology based per 40 CFR 133.102 for BOD, TSS, and pH, and water quality based for ammonia, E. coli, total phosphorus, and WET. Monitoring will continue to be required for total nitrogen and flow. In addition, in keeping with the Kansas Nutrient Management Plan, the permittee is encouraged in the new permit to attain the goals of reducing nutrients to 10.0 mg/L for total nitrogen and 1.0 mg/L for total phosphorus as annual average concentrations. A summary of proposed permit limits is included in Table 4-1. Interim permit limits are the requirements until the plant expansion determined in this report is constructed at which point the final permit limits will go into effect.

The monitoring requirements for the effluent from this facility are consistent with KDHE policy and with requirements of other similar facilities within the State. These monitoring requirements also reflect the best professional judgement of the permit writer considering design and process reliability, and past operational and effluent quality information. This permit has been reissued in accordance with the Basin-wide Permit Planning Procedure, with a sampling frequency of weekly for conventional pollutants and monthly sampling for nutrients, a reporting frequency of monthly, and limits for Biochemical Oxygen Demand, Total Suspended Solids, ammonia, E. coli, and total phosphorus. Ammonia has been assigned both interim and final limits. The monitoring for pH, flow, and nutrients with concentration goals is also included.

Table 4-1 NPDES Permit Limits

	INTERIM PERMIT LIMITS[1]	FINAL PERMIT LIMITS[2]
Biochemical Oxygen Demand (BOD)	Weekly – 40 mg/L Monthly – 25 mg/L	Weekly – 40 mg/L Monthly – 25 mg/L
Total Suspended Solids (TSS)	Mechanical Plant Weekly – 45 mg/L Monthly – 25 mg/L Lagoons Weekly – 120 mg/L Monthly – 80 mg/L	Weekly – 45 mg/L Monthly – 30 mg/L
pH	6.0 – 9.0	6.0 – 9.0
Ammonia (mg/L)	Monthly Average	Monthly Average/Daily Max
January	34.3	13.6 / 34.3
February	34.3	13.6 / 34.4
March	32.5	13.3 / 32.5
April	28.4	9.3 / 28.4
May	22.6	6.1 / 17.4
June	22.6	4.4 / 13.5
July	17.2	3.0 / 7.6
August	14.6	3.5 / 9.7
September	20.2	4.3 / 12.1
October	20.2	7.5 / 20.2
November	32.5	11.4 / 32.5
December	35.8	13.6 / 35.8
E. coli (Colonies/100mL)	April thru October – 1,040 November thru March – 2,000	April thru October – 262 November thru March – 2,000
Total Phosphorus	Monitor	≤ 1.0 mg/L as an annual avg goal ≤ 156.63 ppd annual avg limit
Nitrates (NO_3) + Nitrites (NO_2)	Monitor	Monitor
Total Kjeldahl Nitrogen (TKN)	Monitor	Monitor
Total Nitrogen (TKN + NO_3 + NO_2)	Monitor	≤ 10.0 as an annual avg goal

[1] Interim permit limits are in effect until Mill Creek Regional WWTP expansion is constructed in accordance with JCW's Integrated Management Plan. Values listed are for combined effluent of mechanical and lagoon plants unless noted otherwise.

[2] Final permit limits are to be used as basis of design for the Facility Plan.

5.0 Summary of Findings and Recommendations

This Facility Plan helps provide understanding to JCW with regards to treatment technologies, footprints, operations and economic impacts associated with incorporating nutrient removal facilities and processes as required by KDHE to be implemented as part of an integrated plan. The Integrated Plan incorporates recommended projects and implementation dates into a Consent Order for each JCW facility to be in full compliance with nutrient removal limits. The findings of this Facility Plan will be incorporated into future revisions of the Integrated Plan.

This TM provides recommendations for the flow rates and waste loads under existing and future conditions that will serve as the basis for the process and hydraulic design of future treatment facilities. A summary of findings and recommendations is included below.

5.1 EXISTING AND FUTURE FLOWS

MCR WWTP operates as two parallel treatment trains. One train is an activated sludge system sized to handle flows up to approximately 12 mgd (on an annual average basis) and the other is a lagoon system sized to handle flow in excess of what the activated sludge system can effectively treat. Dry weather flows from 2014 through August of 2019 were analyzed to estimate the current AA and peak flows. Previous reports, historical data extrapolation, and population estimates were utilized to approximate future flows.

The Mill Creek watershed is not currently fully developed, so future growth is expected around 40%. Previously it was believed the ultimate flow in the watershed was 24 mgd, however due to decreases in per capita usage, the current anticipated ultimate growth conditions result in an estimated 21 mgd AA flow. The next improvements project at MCR to incorporate nutrient removal is recommended to be sized for the ultimate growth conditions. To provide the longest useful life of the upgraded WWTP, it is recommended that the ultimate AA flow of 21 mgd be split into 4 trains of 5.25 mgd and all four trains built with the future improvements project. The recommended ultimate MCR peak flow is 6Q, or 126 mgd. A summary of the MCR design flows can be seen in Table 5-1.

Table 5-1 MCR Recommended Design Flows

	DIURNAL LOW AA STARTUP	AA STARTUP	AA ULTIMATE	MAX MONTH	PEAK DAY
MCR Design Flows	6.0[1]	12.0[2]	21.0	31.5	126.0

[1] Historically this is 1/2 of the diurnal high (AA startup)

[2] Flow projection based on Figure 2-3, year 2034 startup, assuming 1% growth

5.2 FLOWS AND WASTE CHARACTERISTICS

Plant data over the past 5 years was analyzed and compared to historical trends. The data set included influent composite samples taking approximately four times per month. It has been a common observation at utilities across the nation that domestic wastewater concentrations are increasing while flows remain the same or increase at a lower rate. National trends show increasing use of water conserving fixtures, which reduces per capita water use but does not impact per capita mass contributions. Historical trends in the influent parameter concentrations were evaluated through linear regression of the dataset over time and an assessment of AA values over 5 years.

Although the national trend is showing increasing wastewater concentrations, after evaluating 5 years of MCR plant data there was not enough of a trend at MCR to include for future concentrations. The data analysis concluded that concentrations at MCR compare very similarly to JCW facilities THC and BRM. A summary of the MCR design concentrations is included in Table 5-2.

Table 5-2 MCR Design Loading Summary

PARAMETER	ANNUAL AVERAGE (AA)		MAXIMUM MONTHLY AVERAGE (MM)		PEAK DAY (PD)
	mg/L	ppd	mg/L	ppd	ppd
BOD	207	36,200	179	47,100	72,400
TSS	280	49,000	240	63,800	98,000
VSS	247	43,200	211	56,200	86,200
TKN	41	7,160	35	9,100	12,500
Ammonia	22	3,860	19	4,920	6,780
Total Phosphorus	4.8	840	4.2	1,100	1,480
Ortho-Phosphate	1.9	340	1.8	460	600

5.3 FUTURE PERMIT LIMITS

In keeping with the Kansas Nutrient Management Plan, JCW will be encouraged in the new permit to attain the goals of reducing nutrients to 10 mg/L for TN and 1.0 mg/L for TP as AA concentrations. JCW is also required to comply with TMDL-Waste Load Allocations (WLA) final limits of 156.63 ppd TP as a 12-month rolling average.

5.4 INTERIM SAMPLING RECOMMENDATIONS

Increased sampling frequency is recommended to provide more accurate influent data for the design wastewater characteristics. Additionally, special sampling to capture the fractions of the carbonaceous, nitrogenous, and phosphorus species is recommended in advance of the project design.

DRAFT

MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 2
Preliminary and Primary Treatment

JCW NO. MCR1-BV-17-12
BV PROJECT 403165

PREPARED FOR

JOHNSON COUNTY KANSAS
Wastewater

SEPTEMBER 17, 2020

BLACK & VEATCH

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Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CANDO	Coupled Aerobic-anoxic Nitrous Decomposition Operation
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEA	Cost Effective Analyses
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
CIPP	Cured-in-place Pipe
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSBR	Continuous Sequencing Batch Reactor
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
D	
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry-weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand

Abbreviation	Meaning
ffCOD	Flocculated Filtered Chemical Oxygen Demand
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
ft	Feet
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
H	
HB	Hallbrook Facility
HDD	Horizontal Directional Drilling
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
Hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration
IC	Internal Combustion
IFAS	Integrated Fixed-Film Activated Sludge
in	Inches
IND	Industrial
INF	Influent

Abbreviation	Meaning
IP	Intellectual Property
IPS	Influent Pump Station
IR	Irrigation Use
IRR	Irrigation
IW	Industrial Water Supply Use
J	
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDHE	Kansas Department of Health and Environment
K_e	Light Extinction Coefficient
kWh	Kilowatt-hour
L	
L	Length, Liter
lb	Pound
LF	Linear Feet
LOMR	Letter of Map Revision
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter
mgd	Million Gallons per Day
min	Minute, minimum
mJ	Millijoules
MLE	Modified Ludzack Ettinger
MLSS	Mixed Liquor Suspended Solids

Abbreviation	Meaning
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow
mmBtu	Million British Thermal Units
MOPO	Maintenance of Plant Operations
mpg	Miles per Gallon
MPN	Most Probable Number
µg/L	Micrograms per Liter
N	
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NFIP	National Flood Insurance Program
NH3-N	Total Ammonia
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
NPV	Net Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow
PE	Primary Effluent
PFE	Primary Filtered Effluent
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow
PIF	Peak Instantaneous Flow
PLC	Programmable Logic Controller

Abbreviation	Meaning
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day
pph	Pounds per Hour
PPI	Producer Price Index
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet-weather Flow
Q	
Q	Flow
R	
RAS	Return Activated Sludge
RAS	
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
Sec	Second, Secondary
SF	Square Foot
SG	Specific Gravity
SLR	Solids Loading Rate
SMP	Stormwater Management Program, Shawnee Mission Park Pump Station
SND	Simultaneous Nitrification/ Denitrification
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station

Abbreviation	Meaning
SRT	Sludge Retention Time
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
TYP	Typical
U	
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VFA	Volatile Fatty Acids

Abbreviation	Meaning
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	Width
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation – Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WLWater LevelWK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

The purpose of this technical memorandum (TM) is to summarize the conceptual design of the preliminary and primary treatment facilities at Mill Creek Regional (MCR) wastewater treatment plant (WWTP). This TM includes a discussion of available preliminary treatment technologies, primary treatment alternative evaluation, design criteria of the selected preliminary and primary treatment technologies, footprint and layouts, capital costs, and operational and maintenance (O&M) costs.

For the primary treatment evaluation, a life-cycle cost analysis was developed. The conceptual cost opinion was developed as a 20-year net present value (NPV), which includes the effects of inflation, time-value of money, and equipment O&M. A triple bottom line (TBL) analysis was then completed as the basis for selection of the primary treatment alternatives for further consideration. Social, environmental and operational criteria were weighted and scored to determine the benefit-cost of each alternative.

This TM is one in a series of technical memoranda for the MCR Facility Plan. Additional treatment processes and site optimization of these treatment facilities will be outlined in future TMs.

1.1 BACKGROUND

Prior to this Facility Plan for MCR, an extensive alternative analysis was done for the Tomahawk Creek (THC) WWTP Expansion. The results of this analysis can be used to inform the planning of the MCR Expansion. THC WWTP is a good comparison because it is a similarly sized facility (19 million gallons per day (mgd) annual average (AA) flow) with similar wastewater characteristics, is owned and operated by JCW, and has actual market costs for treatment technologies provided by a Contractor.

In August of 2014, Johnson County Wastewater (JCW) retained Black & Veatch (BV) for the project definition phase of the THC WWTF Expansion. The primary objective of the project definition phase was to confirm through alternative development and evaluation the optimal, proven treatment strategies throughout the WWTF for nutrient removal to meet current and anticipated future NPDES limits for design flows. Evaluation of these alternatives consisted of utilizing JCW's TBL approach to evaluate non-economic factors in addition to developing capital and operating costs for each alternative. Each treatment process evaluation was presented to JCW, who selected a recommended technology to be carried forward through design and construction.

After the project definition phase, the THC WWTF Expansion was continued into detailed design followed by construction. The construction is scheduled to be completed in 2021. During the detailed design phase some of the selected treatment technologies were reevaluated and eventually revised as part of a value engineering effort. The treatment technologies that were part of the final design and eventually carried into construction serve as a valuable comparison for the MCR WWTP.

From TM 1, the design flows for the MCR WWTP were established as shown in Table 1-1. It should be noted that the preliminary and primary treatment processes will be sized to handle the peak secondary flow, which is three times the design AA flows (3Q).

Table 1-1 MCR Design Flows

	DIURNAL LOW AA STARTUP	AA STARTUP	AA ULTIMATE	MAX MONTH	PEAK SECONDARY	PEAK DAY
MCR Design Flows (mgd)	6.00[1]	12.0[2]	21.0	31.5	63.0[3]	126.0

1. Historically this is 1/2 of the diurnal high (AA startup)
2. Flow projection based on TM 1, Figure 2-3, year 2034 startup, assuming 1% growth
3. Peak secondary capacity is 3 times AA Ultimate (3Q)

1.2 INFLUENT PUMPING AND COARSE SCREENING

The majority of influent flow to MCR WWTP is conveyed to the plant via the 66-inch Mill Creek Interceptor, which enters the plant at the Influent Pumping Station (IPS) where it is coarse screened and pumped to the flow control structure. The IPS coarse screening system includes four mechanically cleaned "climber" type bar screens which remove large trash and debris from the incoming flows to protect the influent pumps. The flow from the IPS discharge to the flow control structure is measured by a magnetic-type flow meter. At the flow control structure, the influent from the offsite pumping stations is combined with the IPS discharge flow, then the combined streams flow by gravity through the downstream unit processes.

The peak wet weather treatment capacity of the existing mechanical MCR WWTP is 24 mgd. The UV disinfection is the limiting factor. When flow exceeds approximately 24 mgd at the IPS, or 34 mgd at the flow control structure, the wet-weather pumps convey the excess flow to the head of the partially mixed aeration cells (Cells 3 and 4), bypassing the flow control structure, grit removal basins, and Completely Mixed Cells 1 and 2. The IPS has two dry weather wetwells and one wet weather wetwell. The wetwells share common walls, and flow in excess of the wetwell capacity is diverted between the wetwells via an opening in the wall. The dry weather pumps were replaced as part of the most recent plant expansion, the wet weather pumps and bar screens are original to the plant. A summary of the IPS equipment is provided in Table 1-2, and a section of the existing IPS is shown in Figure 1-1.

Table 1-2 Existing Influent Pumping Station Summary

COMPONENT	COMPONENT SPECIFICATION
Coarse Screening	Type of Screen: Vertical, Mechanical, Front rake cleaned Number of Screens: 4 Channel Width, ft: 4 Channel Depth, ft: 7 Bar Screen Spacing, in: 3/4 Distance from Operating Floor to Channel Bottom, ft: 53.25 Screen Inclination, deg: 80 Capacity, mgd (per screen): 21 Motor, hp (each): 3

COMPONENT	COMPONENT SPECIFICATION
Screenings Conveyor	Number of Units: 1 Width, ft: 2 Capacity, cf/hr: 15 Motor, hp (each): 3
Dry Weather Pumps	Number of Pumps: 4 Type: Submersible, non-clog Design Rating, gpm (per pump): 5,400 Dry Weather Pumps Firm Capacity, mgd: 23.3 Motor, hp (each): 175 Discharge Piping Diameter, in: 20 Wet Well Volume, cf: 38,900
Wet Weather Pumps	Number of Pumps: 3 Type: Submersible, non-clog Design Rating, gpm (per pump): 13,500 Wet Weather Pumps Firm Capacity, mgd: 39 Discharge Piping Diameter, in: 30 Wet Well Volume, cf: 53,600

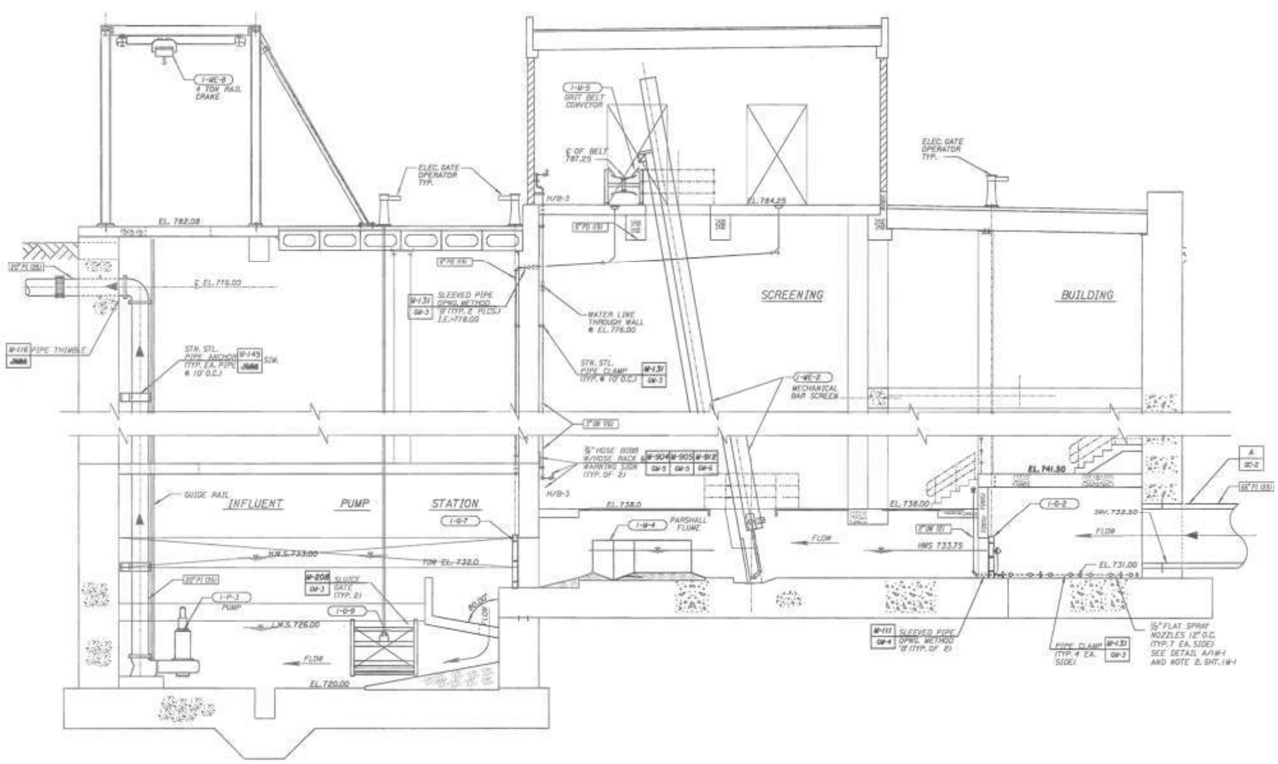


Figure 1-1 Existing IPS Section

As shown in Figure 1-1, the IPS is a deep structure that required difficult excavation. It is believed the IPS facility is in a condition for continued use. Due to the anticipated structural condition at the projected time of the MCR WWTP Expansion, the location on the site, and an attempt to reduce capital costs, it is desirable to reuse the existing IPS. At the point of expected construction, the installed wet weather pumps and coarse screening equipment will be nearing 40 years of age. The dry weather pumps will be approximately 30 years of age. Due to the age of the equipment, it is likely all IPS equipment would be replaced as part of a plant expansion, but the existing IPS structure would be able to be reused. The full extent of IPS improvements will be discussed in TM 9 - Pumping, as the required improvements are impacted by the site layout.

2.0 Preliminary Treatment

Currently, MCR WWTP does not have any fine screening. Flows from the IPS and the offsite pump stations are combined at the flow control structure, and then the combined flows are sent to the grit removal system. The existing grit removal system includes two 18-foot diameter forced vortex grit basins and a grit pumping building. The influent grit channels prior to the vortex basins were modified in the last plant expansion, but the rest of the grit equipment is original to the plant. It is expected that due to the age of the equipment and the current location of the grit removal system, this equipment will be replaced as part of the MCR WWTP Expansion, and the existing grit facilities will be demolished. The new fine screening and grit removal equipment will be located in a new Headworks Building.

Mechanical fine screening removes inorganics and stringy material to protect downstream mechanical devices from excessive maintenance. As treatment technologies have become more advanced, more mechanical devices are in contact with the wastewater. Biological Nutrient Removal (BNR) for example, employs mixers, recycling of flows, submerged media, etc. As a result, BNR treatment is significantly more mechanically intensive when compared to basins that only remove Biochemical Oxygen Demand (BOD), resulting in an increased need for fine screening.

There are a wide variety of fine screening treatment technologies available including climbers, traveling rakes, perforated plates, and drum screens. At a planning stage of design, it is important to understand that there are minimal footprint and economic differences between these different technologies. Consequently, it is recommended to use a 1/4-inch flow through perforated plate fine screen due to its high effectiveness of debris removal and JCW familiarity. The perforated plate fine screen consists of stainless-steel panels, perforated with 1/4-inch holes that form a continuous belt. The wastewater flows through the panels, from front to back, and contaminants are captured on the face of the panel. Lifting tines, pick up the larger objects such as sticks and rocks, from the bottom of the channel. The screenings are transported to the discharge chute, where they are cleaned from the panels by a rotating brush into a washer/compactor. In the washer/compactor, the screenings are washed to remove organics, dewatered, compacted, and discharged to a dumpster below. Design criteria of the fine screening system is outlined in Section 2 of this TM.

The grit removal system will protect the equipment from abrasive material by removing sand, gravel, and other heavy solids from the influent screened wastewater. A grit removal system must also provide for grit conveyance, dewatering (classification), storage, and nuisance control. There are several types of grit removal systems on the market; however, in this region and for this size of treatment facility vortex type, grit removal systems are common. In addition, most JCW facilities are of the vortex grit removal type, so there would be potential benefits to having some consistency across facilities. For these reasons, a vortex type grit removal system is recommended at MCR. Vortex grit removal systems refer to any grit removal technology that uses gravity and centrifugal force to separate the grit particles from the wastewater flow. There are several vortex grit removal technologies. The THC WWTP in-depth evaluation compared the free vortex (Hydro International HeadCell Stacked Tray) system and the forced vortex system.

In the free vortex grit removal systems, or the Headcell, the flow enters the chamber tangentially through flow distribution headers that evenly distribute the influent onto multiple conical trays. A vortex flow pattern is achieved by tangential feed of influent; solids settle into a boundary layer on each tray and are swept down to the center underflow collection chamber. The stacked trays provide for a much shorter settling distance and increased surface areas, which allows for increased removal efficiency of grit particles compared to standard conditions (with gravity alone acting on the grit particle). A schematic of the Headcell grit removal system is shown in Figure 2-1.

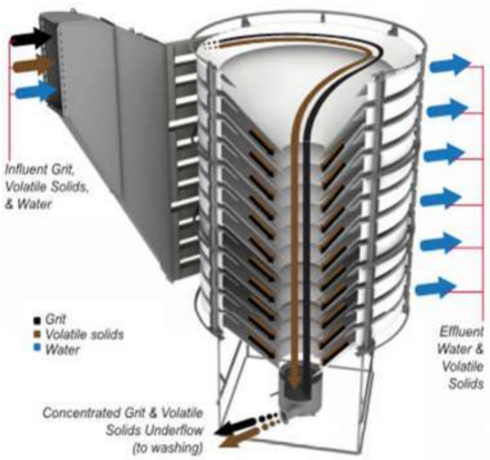


Figure 2-1 Headcell Grit Removal Schematic

Forced vortex grit removal systems are typically of the swirl type, as illustrated by the section cut shown in Figure 2-2. This specific figure is the *Smith & Loveless Baffled Vortex Grit Chamber*. Flow into the swirl-type systems enters tangentially from the outer wall and loses velocity as the flow swirls to the center of the tank. At the center, rotating paddles produce a toroidal flow pattern. The toroidal flow pattern creates a centrifugal force, which increases the settling velocity and allows the increased removal of grit particles compared to the particles being acted upon by gravity alone.

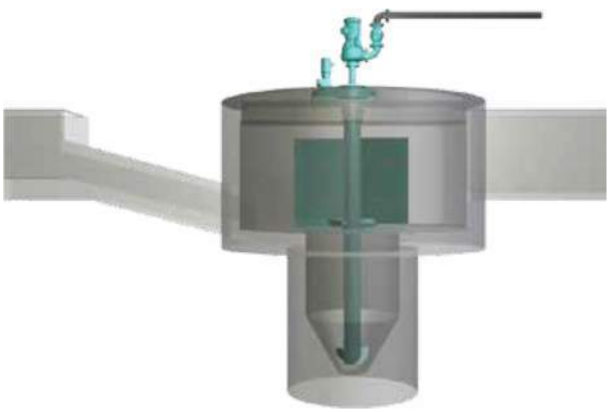


Figure 2-2 Forced Vortex Grit Removal Section

There are several different factors that can be investigated when evaluating which grit removal technology is appropriate, including footprint, capital costs, operation and maintenance costs, ease of expansion, mesh removal size, mesh removal efficiency, and constructability. The biggest difference between these alternatives is the constructability. It has been Black & Veatch's experience that, at this size of facility, the Headcell type grit removal system has constructability benefits affecting the capital cost and construction schedule. Otherwise, at a planning level there is not much to differentiate these two alternatives. Black & Veatch recommends using the Headcell free vortex grit removal system based on constructability benefits and JCW familiarity.

2.1 DESIGN CRITERIA

2.1.1 Fine Screening Design Criteria

Flow entering the Headworks Building will be distributed to any of the three influent channels as presented below in Figure 2-4. Each channel will contain a perforated plate mechanical fine screen with 1/4-inch openings. Table 2-1 summarizes the design criteria for the flow-through fine screening system.

Table 2-1 Fine Screening Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	3 (two duty, one standby)
Maximum Flow Capacity Per Screen, mgd	31.5
Channel Width, ft	5
Installation Angle, degrees	60
Perforation Diameter, in	1/4
Brush Drive Motor, Hp	2
Screen Drive Motor, Hp	2
Number of Sluice Conveyors	1

As seen in Table 2-1, two screens will be capable of screening the peak secondary flow, with a third screen as a standby unit. Screenings collected from each screen will be discharged into a common sluicing trough. A sluice trough uses water to convey the collected screenings to the washer/compactor units. The sluice will be capable of discharging wet screenings into one of two washer/compactor units. One washer/compactor will be sized to handle screenings from all three fine screens. Each washer/compactor will have a manually-actuated knife gate valve to allow it to be isolated from the sluice way. Table 2-2 summarizes the design criteria for the washer/compactors at peak dry weather flow conditions.

Table 2-2 Washer/Compactor Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	2 (one duty/one standby)
Volume Capacity, cf/hr	90
Screenings Volume Reduction, %	60
Screenings Weight Reduction, %	40
Motor, Hp	5
Number of Dumpsters	1

From each washer/compactor unit, screenings will drop into a fabricated stainless-steel hopper with an isolation gate. Typically, screenings will simply drop through the hopper into a dumpster below.

When the dumpster is removed and taken to the landfill, the hopper gate can be closed to hold washed screenings until the dumpster is returned. The dumpster will be sized such that, at annual average conditions, the dumpster could be used over a three-day weekend. At the peak flow, the storage time is reduced.

In order to maintain ideal channel velocities and fine screen hydraulics, it is recommended that the fine screens be operated automatically using flow control set-points. The control narratives for the fine screens would be developed to provide an ideal balance of adequate channel flow velocity while preventing excessive on- and off-cycling of the channels. Each screen would be provided with a PLC-based main control panel provided by the screen manufacturer to control level differential across the screen. Channel water level will be monitored upstream and downstream of each screen. A timer feature will provide a back-up means of screen control. In addition, each washer/compactor will also be provided with a PLC-based main control panel to sequence washer-compactor operations.

2.1.2 Grit Removal Design Criteria

After screening, the wastewater will enter the grit chambers. The grit chambers will be located on the exterior of the Headworks Building. There will be two 12-foot diameter, 12 tray Headcell units. Since they will be located on the exterior of the Headworks Building, all influent and effluent channels — as well as the units themselves — will be covered with checkered plates to capture odorous air. The covers are proposed to allow for venting to an odor control system. Degritted effluent will flow out of the trays, over a weir, and into an effluent trough. To allow the removal of the grit chambers from operation for maintenance purposes, the grit system will be arranged such that effluent from the screens can flow around the Headcell units and discharge directly into the effluent trough via isolation gates. Table 2-3 summarizes the design criteria for the Headcell Grit Removal system.

Table 2-3 Grit Removal Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	2
Trays per Unit	12
Tray Diameter, ft	12
Average Design Flow per Unit, mgd	21
Grit Removal Performance at Average Design Flow	95% removal of 100 micron and larger
Peak Design Flow per Unit, mgd	31.5
Grit Removal Performance at Peak Flow	95% removal of 125 micron and larger

Low flow operation on the grit removal system is a critical consideration, specifically at a facility like MCR where the startup AA is much less than the design AA. Typical grit removal design is based upon a removal efficiency at design peak dry weather flow, with a higher capture rate occurring during low flows. An increased settling of organics is experienced during low flows. It has been Black & Veatch's experience that Headcell free vortex grit systems have difficulty with turndown greater than 10:1. To address this concern, it is recommended to install 2 units each sized for 50 percent capacity. This allows operation of a single unit during average and maximum monthly flow periods while the second unit remains offline or on standby, filled with NPW.

The grit that settles into the collection sump, or the underflow collection chamber, will be continuously pumped from the grit sump to an open vortex grit washing system and dewatering unit. The grit pumps will be housed on the lower level of the Headworks Building, in a chamber directly below the Headcell influent channels, allowing for flooded pump suction under all operation conditions. Each grit slurry pump will be dedicated to a grit washing / dewatering unit that will be located on the upper level. From each grit washer, grit will drop into a fabricated stainless-steel hopper with isolation gate. Typically, grit will drop through the hopper into a dumpster below. When the dumpster is removed and taken to the landfill, the hopper gate can be closed to hold washed grit until the dumpster is returned. Table 2-4 and Table 2-5 summarize the grit pumping and washing / dewatering design criteria.

Table 2-4 Grit Pumping Design Criteria

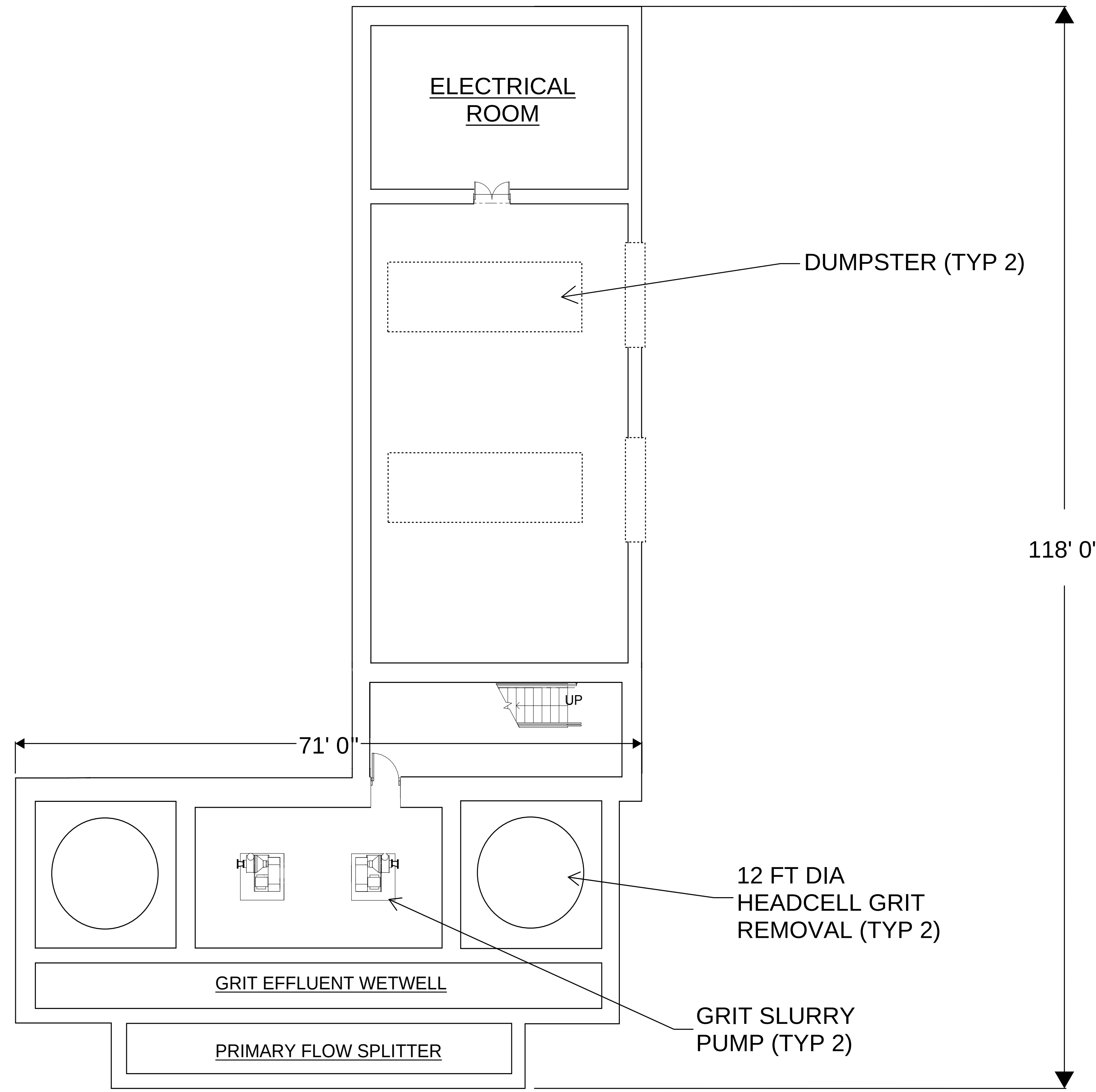
PARAMETER	DESIGN CRITERIA
Number of Units	2
Pump Type	Recessed impeller, centrifugal
Design Flow, gpm	300
Motor Hp	15

Table 2-5 Grit Washing/Dewatering Unit Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	2
Unit Capture Efficiency	95% of grit 75 microns and larger
Design Grit Slurry Flow, gpm	300
Maximum Allowable Grit Slurry Flow, gpm	400
Dewatering Conveyor Motor, Hp	1/3
Maximum Capacity, cy/hr	2

To help maintain ideal channel velocities into the grit removal basins, it is recommended that the basins be operated automatically using flow control set-points. The control narratives for the grit system will be developed to optimize performance while minimizing excessive on and off cycling of basins. Each grit washing / dewatering unit will be provided with a PLC-based main control panel provided by the grit removal system manufacturer / supplier. The panels will both control and sequence the grit pumps and the operational cycle of the washer / dewatering unit.

Refer to Figure 2-3, Figure 2-4, and Figure 2-5 for the proposed layouts of the Headworks Building. Figure 2-4 shows a wet weather meter vault. The preliminary treatment system is designed to handle peak secondary flows; however, to improve the operational control of the wet weather pumps, wet weather piping is routed from the IPS through the wet weather vault to the fine screening influent channels. This allows the wet weather pumps to send flow through preliminary treatment during periods where flow is below 63 mgd. Due to the unpredictable frequency of when wet weather events will cause flows to exceed 63 mgd, having the wet weather pumps with the ability to send flow to the Headworks Building allows these pumps to get routine operation during low flow periods. To protect against sending too much flow through the fine screening influent channel, there is an overflow pipe that sends flow back to the wet weather pumps if the water level gets high enough in the influent channel. This operational functionality should be included in future detailed design.

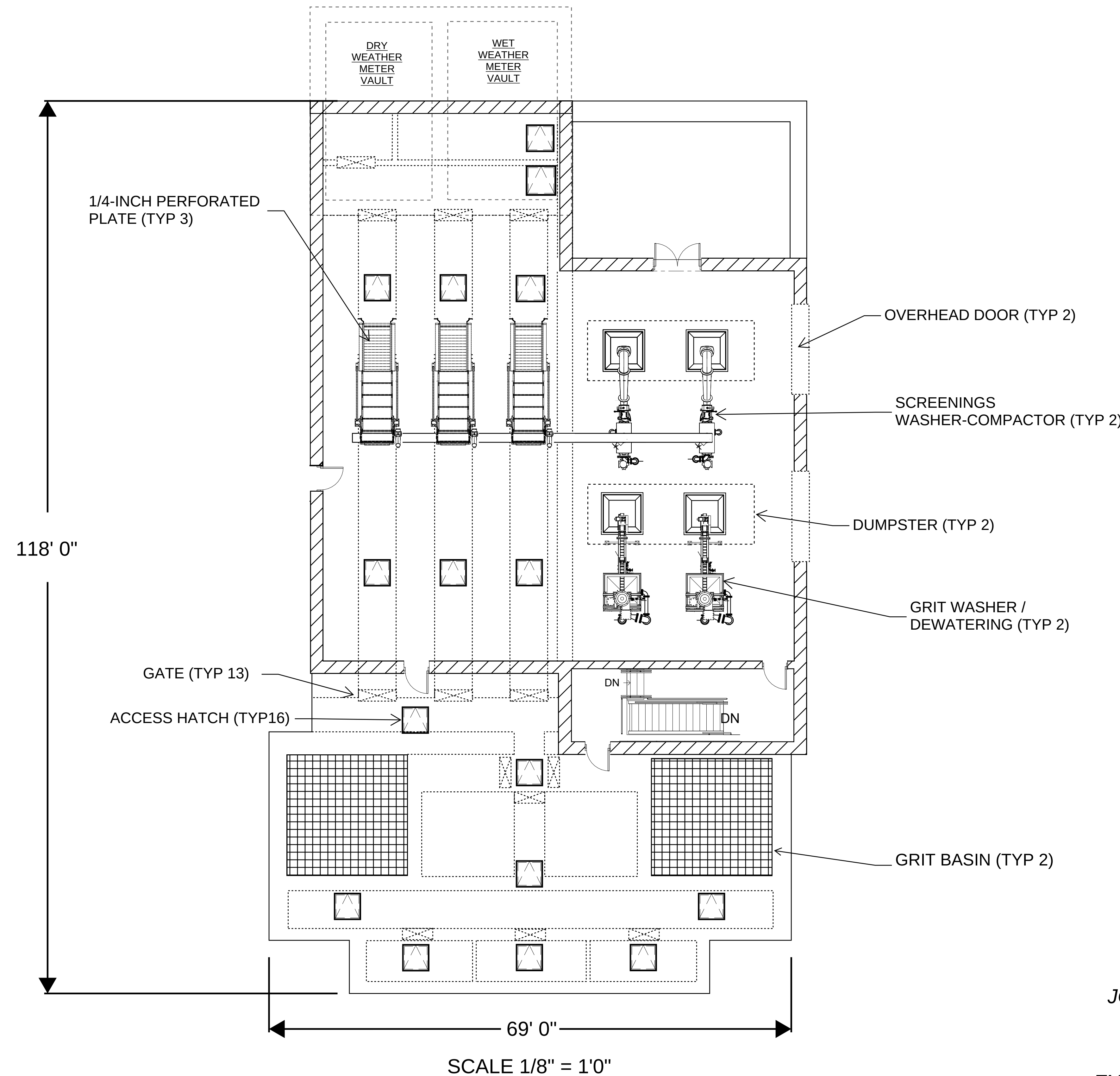


SCALE 1/8" = 1'0"

BLACK & VEATCH

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 2 - Preliminary and Primary Treatment
Headworks Building Lower Level
Figure 2-3



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JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 2 - Preliminary and Primary Treatment
Headworks Building Upper Level
*FIGURE **2-4***

2.2 COST ANALYSIS

Preliminary capital and O&M costs were developed for the fine screening and grit removal systems described in Section 2.1. The estimates are in 2020 dollars.

2.2.1 Summary of Capital Costs

All fine screening and grit removal equipment will be in the Headworks building. An opinion of probable construction cost of this Headworks building is shown in Table 2-6. The costs presented below do not include the cost of electrical, sitework, instrumentation and control, engineering, legal, administration (ELA), or contingencies. These costs will appear as line items in the overall opinion of probable construction cost presented in the Facility Plan Report.

Table 2-6 Screening and Grit Removal Opinion of Probable Capital Construction Cost

	CAPITAL COSTS ($)
Fine Screening Equipment	$605,000
Grit Removal Equipment	$1,051,000
Headworks Building Total Cost[1]	$8,847,000

- Capital costs are presented in January 2020 dollars
- Costs exclude electrical, site, I&C, ELA, and contingency costs
- OPCCs are conceptual level (AACEI Class 4: -15% to -30% low, +20% to 50% high)

The Headworks building at the MCR WWTP was modeled after the Headworks building at the THC WWTP. Both buildings have three screening channels and two Headcell units. The only difference between these two facilities is the Primary Sludge Pump Station. The THC WWTP has a lower level, which houses the five sludge pumps and provides access to piping. Given that the primary treatment technology is being evaluated at the MCR WWTP, it was decided to remove this primary sludge pump station lower level from the MCR WWTP Headworks building. As such, all Primary Sludge Pump Station costs were removed from the cost totals in Table 2-6, including pumps, concrete walls, bridge cranes, etc. These Primary Sludge Pump station costs are included in the Primary Treatment alternatives.

Once the Primary Sludge Pump Station costs were removed from the MCR WWTP Headworks building, the last step in developing the cost was to adjust the cost based on time. The THC WWTP costs are based on 2018 dollars. To represent the most accurate cost at MCR WWTP, these costs have been inflated to 2020 dollars using the Engineering News Record (ENR) construction cost index.

2.2.2 Summary of Operational and Maintenance Costs

Operations and maintenance costs include the cost of power, operating labor, and general maintenance. O&M costs are based on annual average conditions. The power demand is based on the presented design criteria and manufacturer data. The labor costs are based on a BV estimate of hours per week of total labor associated with the Headworks Building. The equipment maintenance cost is based on two percent of the equipment capital cost, which is typical and appropriate at a planning level. The O&M cost summary is presented in Table 2-7.

Table 2-7 Screening and Grit Removal O&M Annual Cost Estimates

	FINE SCREENING	GRIT REMOVAL
Power	$4, 000	$5,000
Labor	$4,000	$7,000
Maintenance	$12,000	$21,000
Chemicals	-	-
Total	**20,000**	**33,000**

3.0 Primary Treatment

Primary clarification serves the purpose of settling suspended solids in the wastewater, primarily organic in nature, that comprises a significant portion of the BOD. This offers advantages to the secondary treatment process by increasing solids removal prior to secondary treatment. Less solids in the secondary treatment corresponds to a smaller footprint and a reduced oxygen demand. A reduced oxygen demand allows for a reduced electricity consumption in the secondary process. Primary treatment also provides more volatile solids in the sludge to subsequently generate biogas for reuse in the digestion process. Disadvantages generally include removal of carbon prior to enhanced secondary treatment that may then need to be supplemented for de-nitrification, and production of primary sludge that is subject to more nuisance conditions than waste activated sludge (WAS). Currently, MCR WWTP does not use primary treatment.

3.1 PRIMARY TREATMENT ALTERNATIVES SUMMARY

For MCR WWTP, three primary treatment alternatives are being evaluated:

- Standard Rate (Traditional)
- Chemically Enhanced Primary Treatment (CEPT)
- Pile Cloth Disk Filters

Standard Rate Primary Clarification is an established technology with predictable performance in total suspended solids (TSS) and BOD removal. It relies on the principal of maintaining a liquid surface overflow rate (SOR) low enough to allow solids to settle to the bottom of the basin, where they are directed by scrapers to draw off pipes for removal. This technology also results in the largest footprint for a given capacity. Figure 3-1 is an example of a Standard Rate Primary Clarifier.

Figure 3-1 Standard Rate (Traditional) Primary Clarifier

CEPT goes a step beyond Standard Rate Primary Clarification and utilizes a coagulant — such as ferric chloride — to improve settleability. This improved settleability allows for a higher surface overflow rate and smaller footprint. Solids particles in the wastewater naturally have a negative ionic surface charge. The coagulant acts to offset this charge, allowing the particles to be attracted to each other so that they coagulate into larger masses, or "floc," which settle more rapidly. CEPT, while well-established and proven, is somewhat less predictable than traditional clarification and requires closer operator control to optimize the chemical feed dosages. JCW operates, and is familiar with, both traditional clarification and CEPT.

There are a few pile cloth disk filters on the market; however Aqua-Aerobics is the only manufacturer that has a filter unit appropriate for a facility the size of MCR. In addition, BV's experience with piloting and design of pile cloth disk filters is only with Aqua-Aerobics. For these reasons, the Aqua-Aerobics Cloth MegaDisk Filters are recommended for evaluation at MCR. The MegaDisk Cloth Filters are a proprietary process offered by Aqua-Aerobic Systems, Inc. shown in Figure 3-2. This filter type has a large installation base for tertiary treatment applications and is an emerging technology for auxiliary wet weather and primary treatment. When used for primary treatment, the solids removal rates are significantly higher than traditional primary clarifiers, resulting in less secondary treatment oxygen demand and more primary sludge. The disk filters have a common influent distribution channel. Flow is directed from this channel to each on-line filter through the means of a filter influent isolation gate. Each filter consists of individual disc segments that are fully submerged under all operating conditions, and they remain static during normal filter operation. Unfiltered flow passes from the outside to the inside of the disk — where filtered flow is collected in the disk's center drum — and conveyed to the effluent channel. The water level through the filter is controlled by an effluent weir.

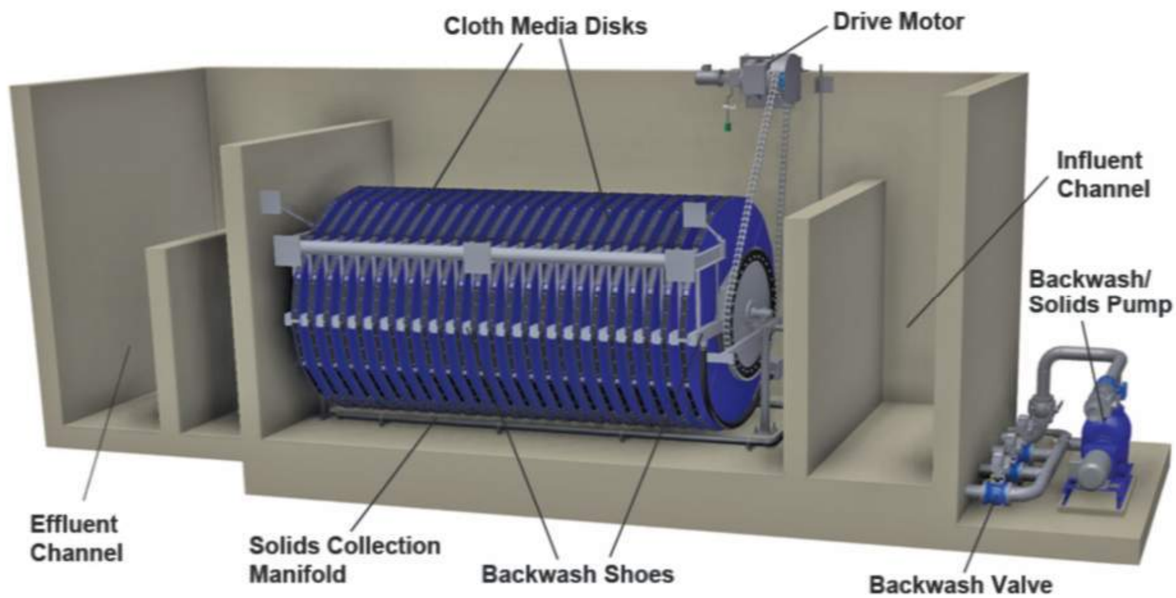


Figure 3-2 Aqua-Aerobic Systems, Inc. MegaDisk Cloth Media Filter

As the filter operates, eventually the filter begins to blind, which initiates a backwash cycle. During the backwash cycle, the disk continues to filter flow and does not go offline. In a primary treatment application, the backwash is directed to a solids thickening process. The system at MCR would be designed such that at peak secondary flows there would be one standby filter cell, that could be placed into service if needed. Due to the modular nature of these filter cells, additional uninstalled spares can be provided by the manufacturer for the event of an equipment failure. Disk filters are the most mechanically intensive and, therefore, the most energy intensive alternative.

3.1.1 Effects of Primary Treatment Selection on Downstream Processes

Disk filters support higher TSS and BOD removal rates than primary clarifiers, which results in a redirection of solids and organic matter to solids processing rather than secondary treatment. The annual average solids loading in the major process streams are provided in Table 3-1.

Table 3-1 Solids Loads to Major Process Units

	TSS	
	Primary Clarifiers[1]	Disk Filters
Primary Treatment		
Influent, ppd	49,000	49,000
Removal, %	50	80
Primary Sludge, ppd	24,500	39,200
Primary Effluent, ppd	24,500	9,800
Gravity Thickener/Fermenter[2]		
Influent, ppd	24,500	39,200
Capture, %	85	85
Thickened Primary Sludge, ppd	20,850	33,300
Overflow, ppd	3,650	5,900
WAS DAF Thickener		
WAS, ppd	16,000	10,900
Capture, %	95	95
Thickened WAS, ppd	15,200	10,400
DAF Effluent, ppd	800	500
WASStrip Process[2]		
Primary Sludge Feed, ppd	1,050	1,650
WAS Feed, ppd	15,200	10,400
Effluent, ppd	16,250	12,050
WASStrip DAF Thickener		
Thickener Feed, ppd	16,250	12,050
Capture, %	95	95
WASStrip Solids, ppd	15,450	11,450
DAF Effluent, ppd	800	600
Anaerobic Digester		
Primary Sludge Feed, ppd	19,800	31,650
Thickened WASStrip Solids Feed, ppd	15,450	11,450
Digester Feed, ppd	35,300	43,100

	TSS	
	Primary Clarifiers[1]	Disk Filters
Digester Feed Solids, %VS	82	85
VS Destruction[3], %	45	50
Digester Solids, ppd	22,200	24,850
Gas Yield, cubic yards / day	7,250	10,150
Dewatering		
Centrifuge Feed, ppd	22,200	24,850
WAS DAF Thickener Capture, %	98	98
Cake Solids, ppd	21,750	24,350
Centrate Solids, ppd	450	400

Notes:
1. At AA flows, traditional clarifiers and CEPT have the same solids removal rate.
2. Solids destruction by fermentation in the gravity thickener and WASStrip process was assumed to be null to conservatively size equipment.
3. The fraction of biodegradable VS is assumed to be greater for the disk filter option due to a higher contribution of primary solids.

At a high level, downstream impacts due to increased primary solids removal by disk filters include the following:

- **Secondary Treatment.** Disk filters send less solids and organic matter to the aeration basins, thereby requiring less secondary treatment basin volume and secondary clarifier capacity. By reducing the organic load to the secondary aeration basins, the aeration demand is lowered, and energy savings are achieved compared to secondary treatment using traditional primary clarification.
- **Primary Sludge Thickening and Fermentation**. If primary clarifiers are used at MCR, primary sludge would be thickened and fermented in a gravity thickener/fermenter to produce volatile fatty acids (VFA) for biological phosphorus removal. Although there are several ways to handle thickening of the primary solids, a gravity thickener/fermenter was selected at MCR due to the operational benefits and control of VFA production. Disk filter backwash is significantly more dilute than primary sludge, with the backwash solids concentration expected to be approximately 0.1-0.3 percent (compared to approximately 0.5-1 percent for primary sludge). Directing all backwash flow to the gravity thickener/fermenter would result in significantly higher overflow rates and a dilute VFA concentration. These effects would be detrimental to RAS fermenter performance, that depends on little turbulence and high VFA concentrations to create conditions suitable for phosphorus release; therefore, the backwash water requires an intermediate thickening step with the thickened backwash solids sent to the gravity thickener/fermenter. A schematic of the disk filter sludge thickening process is presented in Figure 3-3. For this costing effort, the backwash thickening technology was selected as gravity thickeners for their ease of operation. Technologies such as belt filter presses and rotary drum thickeners are also viable alternatives. Thickening of backwash solids is an ongoing research topic for

Aqua-Aerobics, who is currently evaluating multiple thickening technologies for reducing the volume of backwash water.

The disk filters lower the BOD load sent to the activated sludge basins, but disk filters result in more VFA being produced in the gravity thickener/fermenter. The disk filter therefore builds in a degree of control over the type of carbon that is sent to the secondary basins. VFA is highly desirable as it supports reliable biological phosphorus removal.

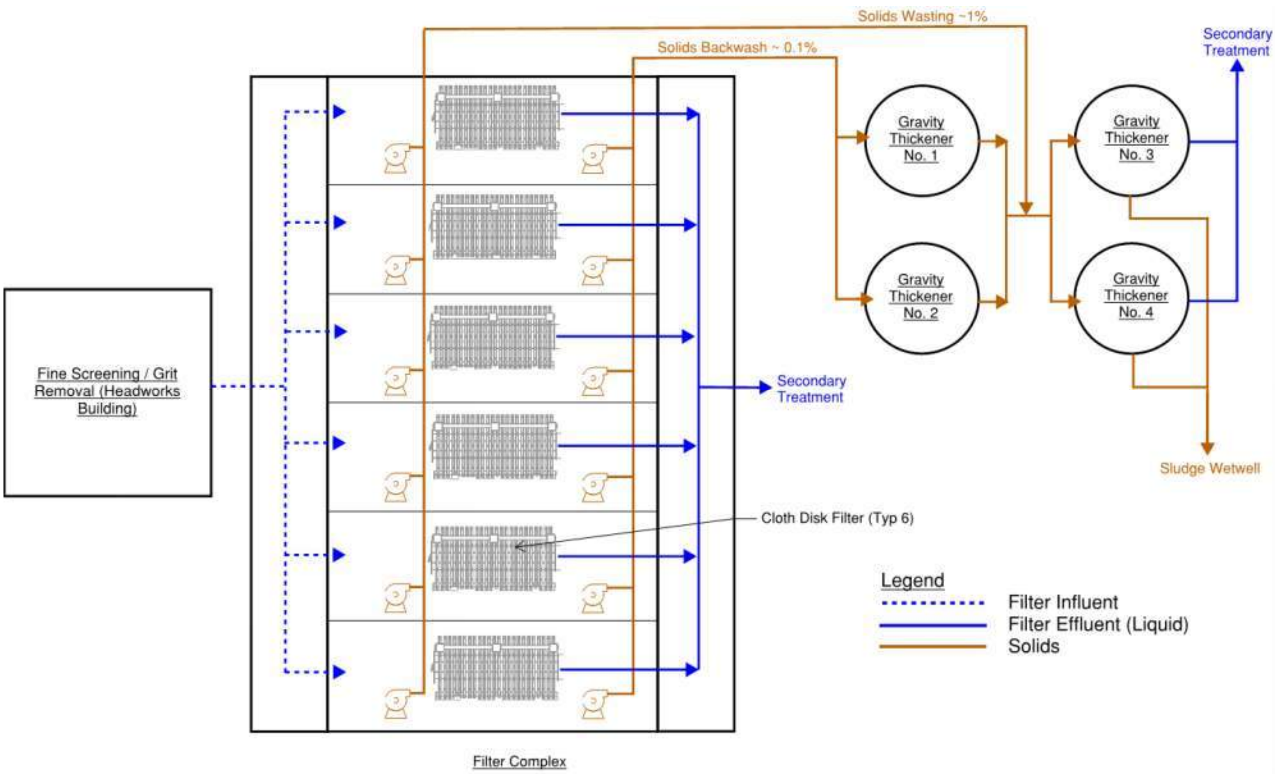


Figure 3-3 Disk Filter Primary Sludge Thickening Schematic

WAS Thickening and Phosphorus Recovery (WASSTRIP). Less WAS is produced with disk filters; therefore, the DAFs and WASSTRIP basin are smaller. Secondary sludge, or WAS, thickening is discussed in TM 6 – Biosolids Treatment.

- **Anaerobic Digestion**. The sludge blend tank and anaerobic digester increase in size due to the slightly higher solids and hydraulic loading by the disk filter solids stream. With disk filters sending more biodegradable solids to the digester, VSS destruction and gas yield is increased. There are also more solids to the dewatering process; however, it is assumed that increased dewatering operation can make up for the increase in solids. Anaerobic Digestion is discussed in TM 6 – Biosolids Treatment.
- **Sidestream Treatment.** The sidestream equalization basin is expected to increase in size minimally. The size of the sidestream process units is not expected to differ significantly. Sidestream Treatment is discussed in TM 3 – Secondary and Sidestream Treatment.

3.2 DESIGN CRITERIA

Each of the primary treatment alternatives will be designed to treat flows ranging from the startup Diurnal Low Flow of 6 mgd to the Peak Secondary Flow of 63 mgd, as presented in Table 1-1.

3.2.1 Alternative 1 – Traditional Primary Clarifier Design Criteria

Traditional Primary Clarification has many benefits. The most appealing aspect is that it is a proven treatment technology and JCW has lots of familiarity with operation. The biggest downside is this alternative requires the largest footprint. The design criteria for Alternative 1 is shown in Table 3-2.

Table 3-2 Traditional Primary Clarifiers Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	4
Diameter, ft (each)	115
Max Surface Overflow Rate, gpd/SF	1,500
Min Hydraulic Retention Time (HRT), days	1.70
Surface Area, SF	10,400
Side Water Depth (SWD), ft	14
Tank Volume, cf	145,400
Total Footprint Required, SF	42,000
Anticipated BOD Removal, %	40
Anticipated TSS Removal, %	50
Clarifier Drive Motor, hp	0.5

3.2.2 Alternative 2 – Traditional Primary Clarifiers with CEPT Design Criteria

CEPT is very similar to traditional primary clarification outlined in Alternative 1. In fact, Alternative 2 operation would be the same up to 38 mgd. When flows exceed 38 mgd, the maximum SOR is achieved, so ferric chloride would be added to increase the settleability of the solids, allowing for a higher SOR; therefore, the biggest benefit of Alternative 2 is a reduced footprint. The higher SOR at peak flow conditions allows for the reduction of an entire clarifier unit. The downside of this Alternative is JCW staff would have to handle chemicals and more solids during high flow events. If this Alternative is selected, the ferric chloride storage and feed system will be stored in a separate building, along with operation of the chemical feed for CEPT. The design criteria for Alternative 2 is shown in Table 3-3.

Table 3-3 Traditional Primary Clarifiers with CEPT Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	3
Diameter, ft (each)	105
Max Surface Overflow Rate, gpd/SF	2,400
Min Hydraulic Retention Time, days	1.05
Surface Area, SF	8,700
Side Water Depth, ft	14
Tank Volume, cf	121,200
Total Footprint Required, SF	26,000
Anticipated BOD Removal, %	40
Anticipated TSS Removal[1], %	80
Clarifier Drive Motor, hp	0.5
Note: 1. Peak removal achieved after chemical addition	

For both Alternative 1 and 2, the primary clarification equipment would be of the plow blade scraper type and the basins would be provided with flat aluminum basin covers for odor control. Flow would enter through the center column and flow over v-notched weirs at the perimeter of the basin to the effluent trough, then be conveyed to the secondary treatment process. The clarifiers would be controlled by Start and Stop selections made by the operator. When in service, the clarifier mechanisms would be operated at a constant speed.

3.2.3 Alternative 3 – Cloth Disk Filters Design Criteria

The primary disk filters used in Alternative 3 are significantly different than the technology used in Alternatives 1 and 2. The design of filters is based on the solids loading rate (SLR) and hydraulic loading rate (HLR). Although disk filters are a less traditional technology for Primary Treatment, there are still significant benefits. One benefit is an increased removal rate across all flow ranges. Based on discussion with Aqua-Aerobics, it is estimated that Alternative 3 could achieve an 80 percent removal rate. An 80 percent removal rate would result in increased primary sludge and less secondary aeration demands, while also having a significantly smaller footprint. The design criteria for Alternative 3 is outlined in Table 3-4.

Table 3-4 Cloth Disk Filters Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Cells	6
Number of Disks per Cell	24
Diameter per Disk, ft	10

PARAMETER	DESIGN CRITERIA
Filter Unit Submerged Filtration Area, SF	2,582
Total Filter Area, SF	15,492
Max Hydraulic Loading Rate[1], gpm/SF	3.39
Max Solids Loading Rate[1], ppd/SF	7.61
Total Footprint Required, SF	6,200
Anticipated BOD Removal, %	50
Anticipated TSS Removal, %	80
Filter Drive Motor, hP	5
Note: 1. Max loading rates are with one cell out of service.	

The disk filters are designed to achieve the recommended HLR and SLR at peak conditions with one cell out of service. As each operational filter cell begins to blind and the level within each filter bay rises to a preset point, a PLC automatically initiates a filter backwash cycle that includes rotating the disk assembly and activating the suction style backwash pumps that remove filtered material from the outside of each disk.

To bring additional filter cells on-line to match the influent flow conditions, the influent isolation gate will be opened automatically by the PLC control system based on a flow set point and the filter will commence filtering flows automatically. This control can also be tied to the filter's internal high-level alarm. Each filter is equipped with an overflow weir that will automatically bypass unfiltered flow and combine it with filtered flow to maintain continuous operation.

3.2.4 Primary Sludge Projections and Pumping

Regardless of which Primary Treatment alternative is selected, primary sludge and scum will be produced and then pumped to the solids processing facilities. Primary sludge from Alternatives 1 or 2 would be a typical primary sludge with anticipated concentration up to 1 percent. Primary sludge from Alternative 3 would be much thinner, approximately 0.1 to 0.3 percent. It should also be noted that, although peak removal percentages for Alternative 2 and 3 are both 80 percent, CEPT has a higher primary sludge production due to the addition of iron, which precipitates to form more primary sludge; however, this does not affect facility sizing because clarifiers only operate in CEPT mode beyond max month flows. Since this evaluation is focused on AA conditions, downstream equipment sizing is not affected. Scum projections vary depending on the size and wastewater characteristics of a treatment plant. It is estimated that MCR would produce around 6,000 gallons per day (gpd) of scum for all primary treatment alternatives. Table 3-5 summarizes the primary sludge projections for all primary treatment alternatives.

Table 3-5 Primary Sludge Projections

FLOW CONDITION	ALTERNATIVE 1		ALTERNATIVE 2		ALTERNATIVE 3	
	PS, PPD	Flow[1], gpm	PS, PPD	Flow[1], gpm	PS, PPD	Flow[2], gpm
Annual Average (21 mgd)	24,520	408	24,520	408	39,231	1,896
Max Month (31.5 mgd)	31,919	532	31,919	532	51,071	2,844
CEPT Threshold (38 mgd)	49,000	816	78,400	1,306	78,400	3,430
Peak Secondary Flow (63 mgd)	49,000	816	78,400	1,306	78,400	5,688

Notes:
[1.] 1% solids might be achieved; however, solids projections are based on 0.5% to provide safety factor.
[2.] 0.3% solids might be achieved; however, solids projections are based on 0.2% to provide safety factor.

For Alternatives 1 and 2, typical primary clarifier design includes sludge removal from the bottom of the clarifier using the collector mechanism that scrapes the sludge towards the center sludge hopper and drawoff pipe. From there, the primary sludge pumps send solids to the downstream solids processing facilities. The primary sludge pumps would be the flooded suction type and, for Alternatives 1 and 2, would be in a stand-alone Primary Sludge Pump Station, as shown in Figure 3-3. The Primary Sludge Pump Station will be a two-level structure, with pumps on the lower level and a masonry electrical room above. The primary sludge pumps will be provided with an adjustable frequency drive (AFD) and flow meter. The pump speed will be adjusted manually or automatically based on maintaining an operator selected flowrate. The operator selected flowrate is based on maintaining an average sludge blanket depth. The Primary Sludge Pump Station lower level would also house the scum pumps. Scum and other floating debris would be removed from the surface of the clarifiers using full radius skimmers, which direct the scum to a scum beach. The full radius skimmers then eventually direct the scum to the scum pit located in the lower level of the Primary Sludge Pump Station. The lead scum pump will operate when the level in the scum pit reaches the high level setpoint and will stop when the scum pit reaches the low-level set point. The pump speed will be manually set by the operator.

All pumps will be the same model to provide additional levels of redundancy. In addition, the standby primary scum pump can be used as a swing pump that can serve either application (sludge or scum), thus providing firm capacity for primary sludge and scum. Primary sludge will be pumped to the Gravity Thickeners while primary scum will be pumped to the sludge blend tank prior to digestion. The lower level of the Primary Sludge Pump Station will be equipped with a bridge crane, which will allow the pumps to be moved to a designated location where they can be lifted through an above access hatch. A summary of the primary sludge and scum pumps is provided in Table 3-6.

Table 3-6 Alternatives 1 and 2 Primary Sludge and Scum Pumps Design Criteria

PARAMETER	DESIGN CRITERIA
Primary Sludge Pumps	
Number of Units	(3 duty + 1 shared swing)
Type	Progressive Cavity, Direct Drive
Drive Type	Adjustable Frequency Drive (AFD)

PARAMETER	DESIGN CRITERIA
Max Pump Capacity, gpm	225
Motor Rating, hp	25
Primary Scum Pumps	
Number of Units	(1 duty + 1 shared swing)
Type	Progressive Cavity, Direct Drive
Drive Type	Adjustable Frequency Drive (AFD)
Max Pump Capacity, gpm	225
Motor Rating, hp	25

For Alternative 3, influent wastewater flows from the influent channel into each filter cell over the influent weir, completely submerging the static cloth media disks. As influent passes through the cloth on both sides of the disk, solids accumulate on the pile cloth media and a solids mat is formed. Filtrate is collected in the center tube, then directed to the effluent chamber and over the effluent weir. During backwash, the filters remain in service. One-third of the disks are backwashed at a time by rotating the entire filter assembly. Solids are vacuumed from the surface by backwash shoes that pull filtered water from inside the filter disk. The backwash shoes make firm contact with the cloth media, maximizing effective cleaning while filtration continues on two-thirds of the disks without interruption. Due to the vertical orientation of the cloth media, heavier primary solids settle to the bottom of the tank. These solids are pumped on an intermittent basis by opening a valve and using each filter cell's solid wasting pump (provided by the manufacturer). The backwash and solids wasting pumps send flow to the gravity thickeners. For redundancy purposes, the backwash pump and the solids wasting pump are identically sized. A summary of the manufacturer provided disk filter pumps is outlined in Table 3-7.

Table 3-7 Alternative 3 Primary Sludge Pumping Design Criteria

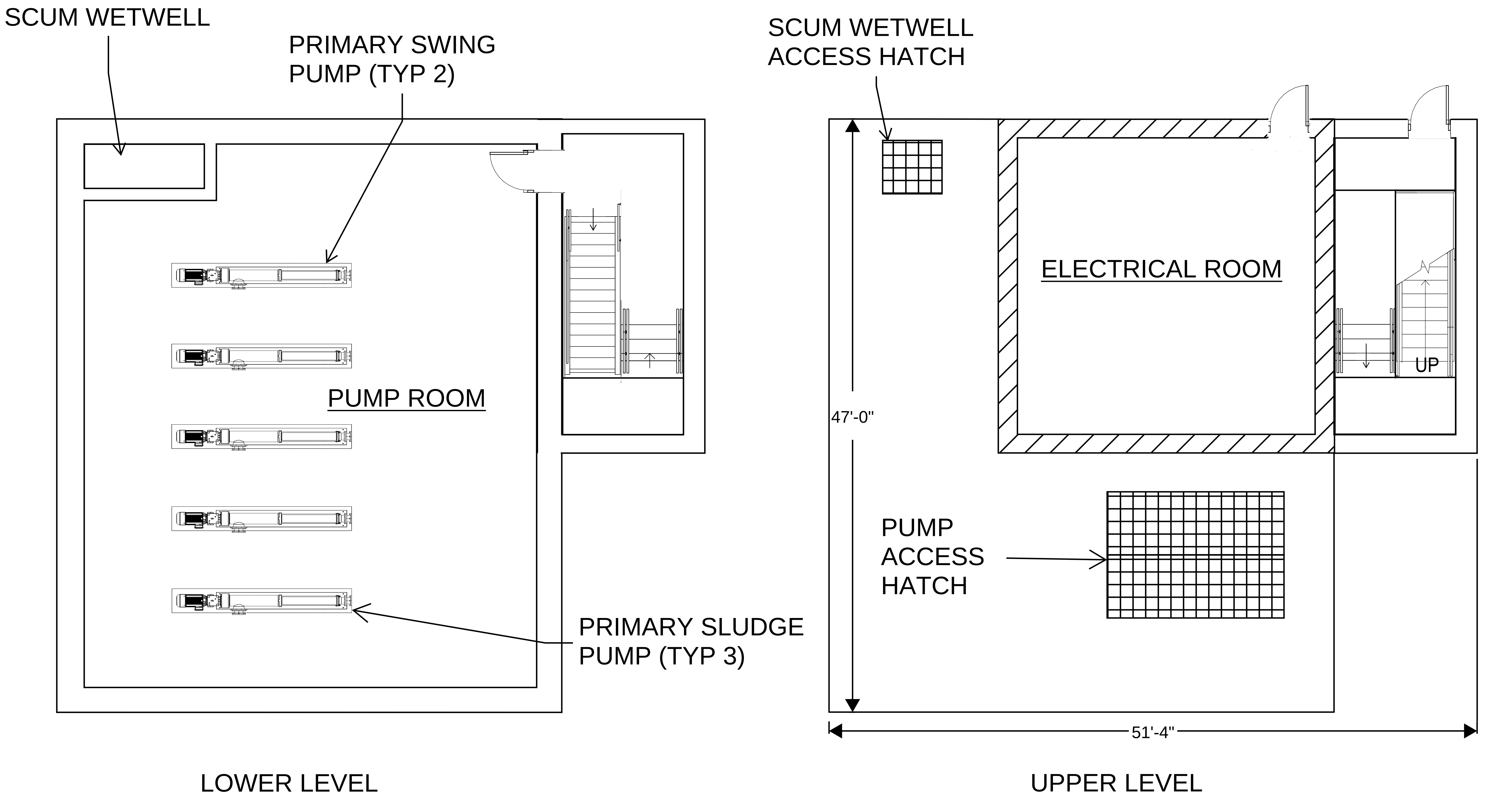
PARAMETER	DESIGN CRITERIA
Number of Pumps	12 (1 backwash and 1 solids wasting pump per filter cell)
Type	Self-priming centrifugal
Pumping Capacity, gpm	780
Pump Motor, hp	20

Scum removal for the primary disk filters is individual to each filter. Scum is removed when it flows over the scum weir. It is assumed that the combined scum from each filter unit would be sent to the plant drain and returned to the head of the plant. If the plant drain is not an option, a common scum wetwell with submersible pumps could be provided to send the primary scum to the solids processing facilities. In total, each filter cell consists of a filter drum and motor, backwash pump and motor, and solids wasting pump and motor; all will be provided by Aqua-Aerobics.

3.2.5 Layout and Footprint Considerations

As previously mentioned, the primary sludge will be pumped with progressing cavity sludge pumps located in the lower level of a stand-alone Primary Sludge Pump Station in Alternatives 1 and 2. In addition, two equally sized scum pumps will also be provided in the lower level of the Primary Sludge Pump Station. The upper level of the Primary Sludge Pump Station is a masonry electrical building for the pumps below. A layout of the Primary Sludge Pump Station is shown in Figure 3-3.

In Alternative 3, each filter cell is provided with a filter drum, a backwash pump, and a solids wasting pump. The facility is essentially three pairs (modules) of two filter cells that share a filter pump station. The top level of each module is a masonry electrical room with equipment for the two filter cells. In total at MCR, there are three filter modules constructed together to form the Primary Disk Filter Complex, as shown in Figure 3-2, Figure 3-3, and Figure 3-4.

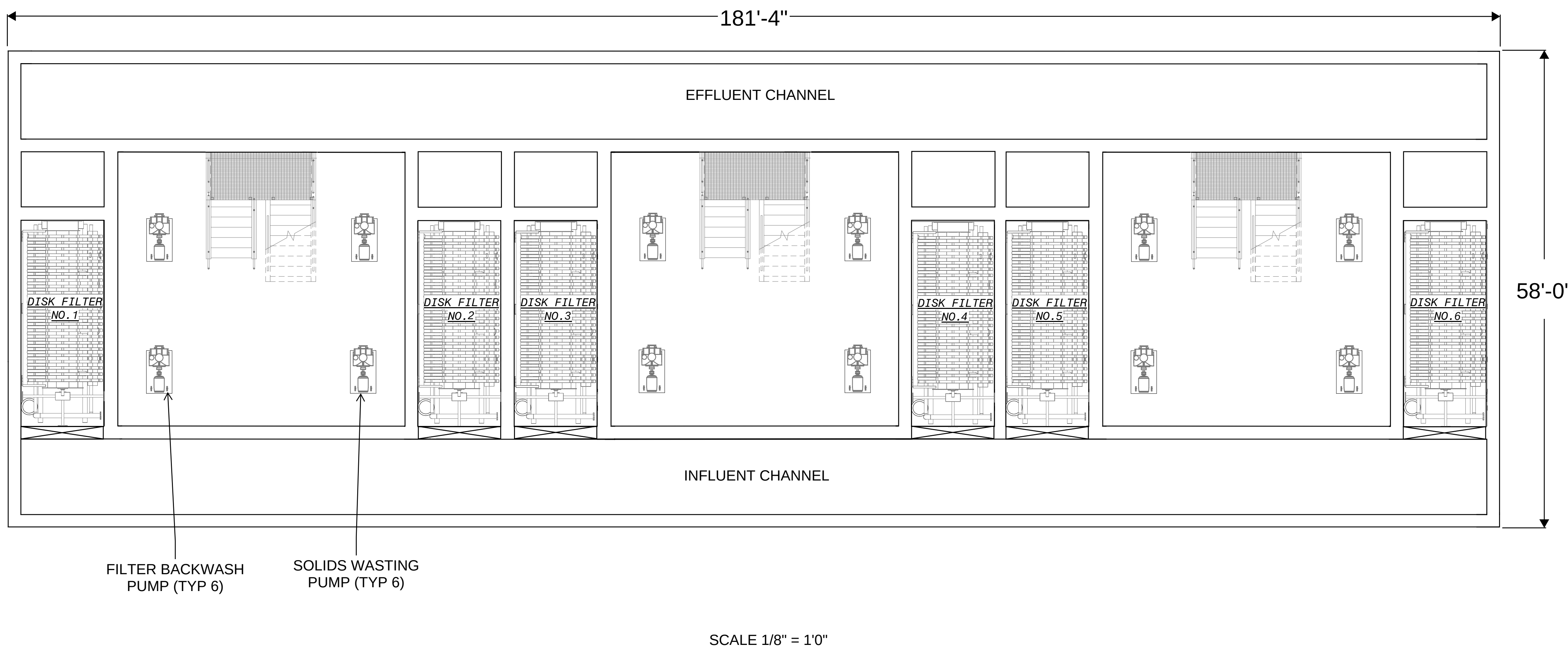


SCALE 1/4" = 1'0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

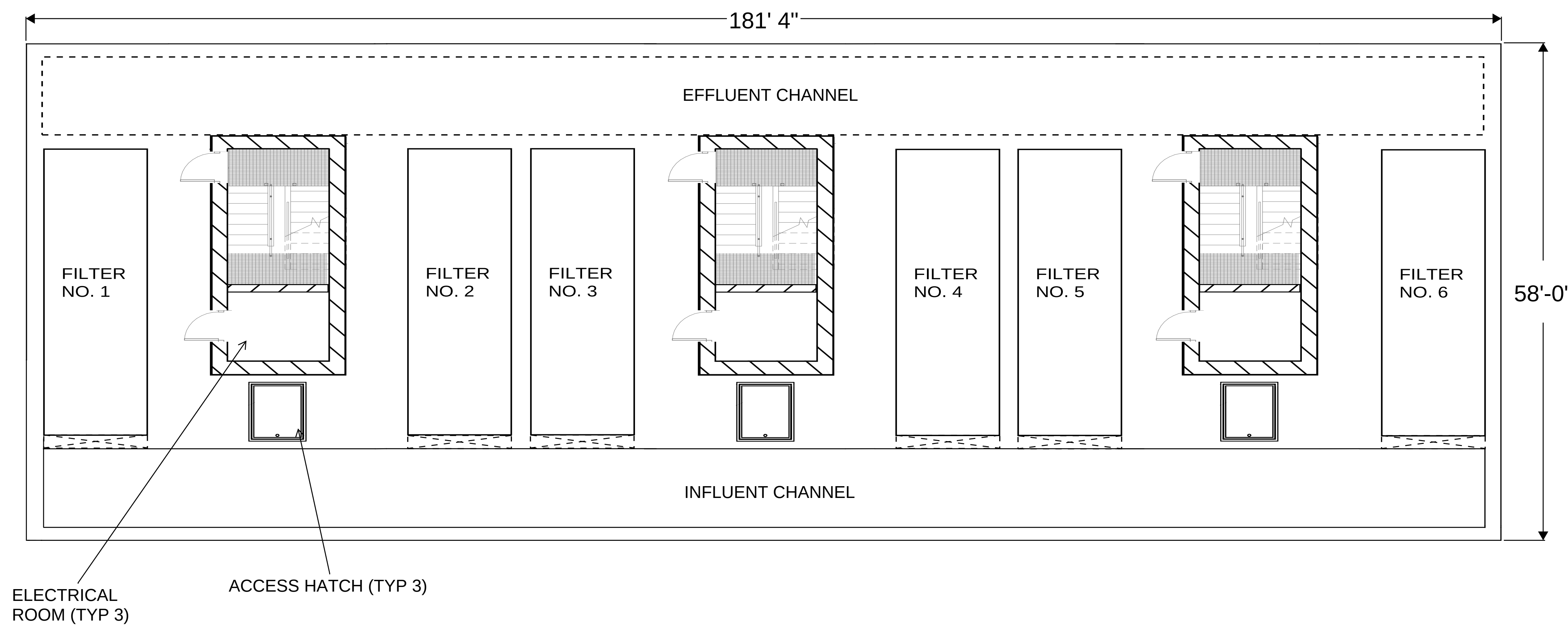
TM No. 2 - PRELIMINARY AND PRIMARY TREATMENT
PRIMARY SLUDGE PUMP STATION
FIGURE 3 - 4

BLACK & VEATCH



BLACK & VEATCH

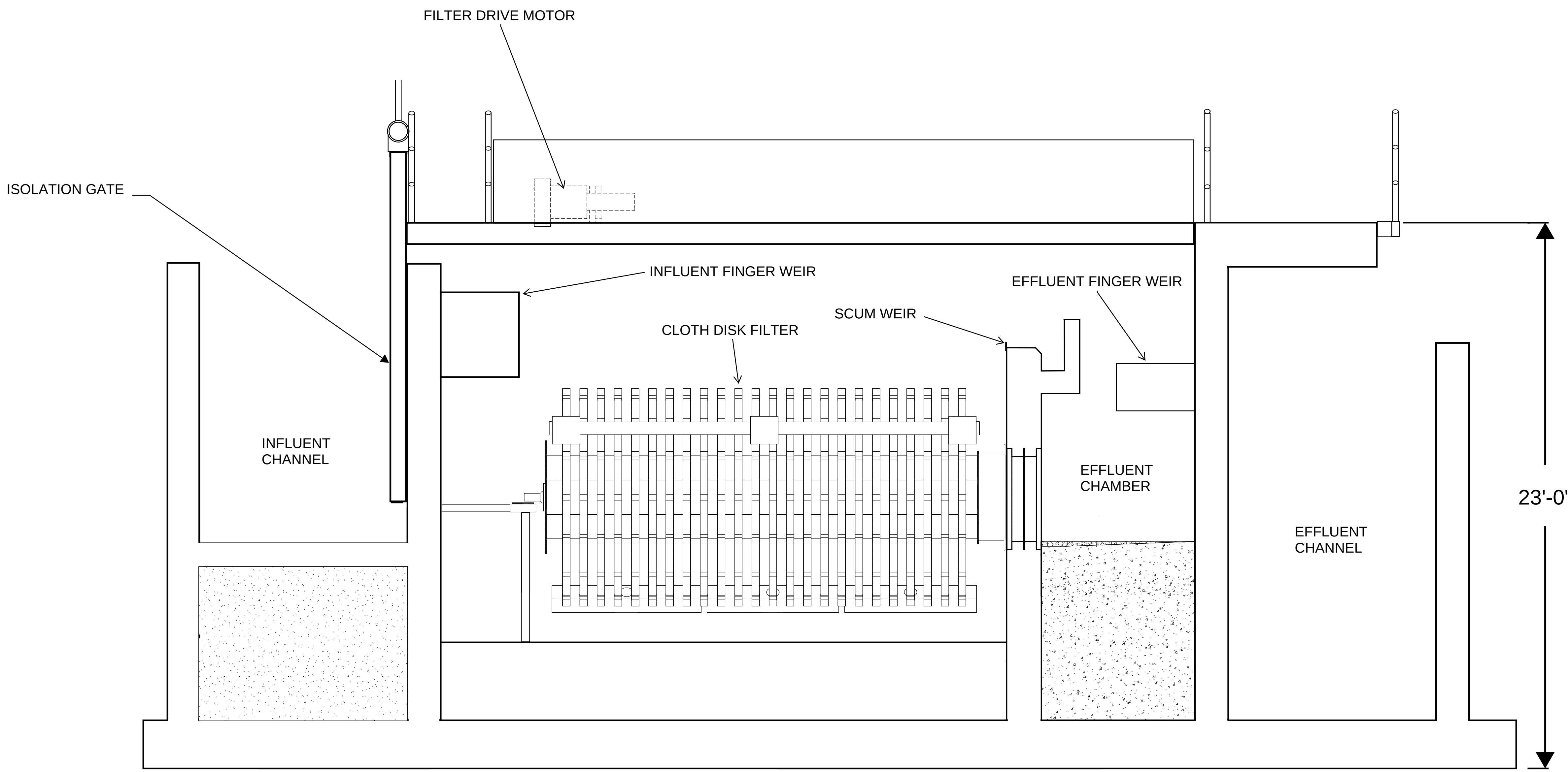
JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN
TM No. 2 - PRELIMINARY AND PRIMARY TREATMENT
PRIMARY DISK FILTERS LOWER LEVEL
FIGURE 3-5



SCALE 1/8" = 1"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 2 - PRELIMINARY AND PRIMARY TREATMENT
PRIMARY DISK FILTERS UPPER LEVEL
FIGURE 3-6



SCALE 3/8" = 1' 0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 2 - PRELIMINARY AND PRIMARY TREATMENT
PRIMARY DISK FILTERS SECTION
FIGURE 3-7

3.3 COST ANALYSIS

Preliminary capital and O&M costs were developed for each of the Primary Treatment Alternatives described in Section 3.2. The basis of design presented in this TM was used to develop a preliminary opinion of probable construction cost for the primary treatment alternatives. The costs presented below do not include cost of electrical, sitework, instrumentation and control, ELA, or contingencies. These costs will appear as line items in the overall opinion of probable construction cost presented in the Facility Plan Report. The estimates are in 2020 dollars.

3.3.1 Summary of Capital Costs

An opinion of probable construction cost (OPCC) for each alternative is shown in Table 3-8.

Table 3-8 Primary Treatment Alternatives Capital Cost Summary

ALTERNATIVE	CAPITAL COST
Alternative 1 – Traditional Clarifiers	$10,438,000
Alternative 2 – Traditional w/ CEPT	$7,685,000
Alternative 3 – Cloth Disk Filters	$11,401,000

- Costs for a Primary Sludge Pump Station are included for Alternatives 1 and 2.
- Capital costs are presented in January 2020 dollars.
- Costs exclude electrical, site, I&C, ELA, and contingency costs.
- OPCCs are conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

The difference in capital cost between Alternatives 1 and 2 is primarily due to Alternative 1 having an additional clarifier. Alternative 2 costs also include costs associated with the chemical feed equipment required for CEPT. Alternative 3 has the highest capital cost primarily due to a significant increase in equipment costs. The equipment costs for Alternative 3 are approximately nine times the equipment costs of Alternative 1, if the sludge pump station equipment is also included with Alternative 1 costs. When compared to Alternatives 1 and 2, Alternative 3 has a lower concrete cost due to a smaller associated footprint.

3.3.2 Summary of Operational and Maintenance Costs

Operations and maintenance costs include the cost of power, chemicals, operating labor, and general maintenance. O&M costs are based on annual average conditions and solids production. The power demand is based on the presented design criteria and manufacturer data. The labor costs are based on a Black & Veatch estimate of hours per week of total labor associated with each alternative. The equipment maintenance cost is 2 percent of the equipment capital cost. The O&M costs for each alternative are shown in Table 3-9.

Table 3-9 Primary Treatment Alternative O&M Cost Summary

	ALTERNATIVE 1 – TRADITIONAL CLARIFIERS	ALTERNATIVE 2 – TRADITIONAL W/ CEPT	ALTERNATIVE 3 – DISK FILTERS
Power	$ 17,000	$ 16,000	$ 29,000
Labor	$ 53,000	$ 49,000	$ 107,000
Equipment Maintenance	$ 15,000	$ 12,000	$ 131,000
Chemicals	-	$ 19,000	$ 11,000
Total	**$ 85,000**	**$ 96,000**	**$ 278,000**

There are a few differences between annual O&M costs for Alternatives 1 and 2. Alternative 1 has an additional clarifier; therefore, it has a slight increase in associated annual power, labor, and equipment maintenance, and the cost increase associated with the chemicals used in Alternative 2 results in a net increase of $11,000 per year. The annual O&M costs for Alternative 3 are significantly more when compared to Alternative 1 and 2. This significant increase is primarily due to two factors: labor and equipment maintenance. The disk filters in Alternative 2 have 6 cells with a total of 14 pumps, whereas Alternative 1 has 4 units with a total of 5 pumps. This increase in number of units is expected to result in an annual labor cost of $107,000. The equipment maintenance cost is a fixed 2 percent of the equipment capital cost. The filter modules have a smaller footprint than Alternatives 1 and 2, but have a significant increase in equipment capital cost. This estimated equipment maintenance cost over five years is representative of the cost of replacing the filter media. According to the manufacturer, the recommended filter replacement in a tertiary application is seven years. It is estimated that, in a primary treatment application, this replacement period would be accelerated due to an increased loading. Lastly, there is a small annual chemical cost associated with using Disk Filters. A small amount of sodium hypochlorite is periodically applied to clean the filters. Hypochlorite is applied as a preventative maintenance measure to minimize buildup on the cloth media filters.

3.3.3 Adjustments to Capital and O&M Costs

As discussed in Section 3.1.1, the primary treatment selection affects downstream processes. Although selections of downstream process are outside the scope of this TM and will be discussed in future TMs, the required treatment processes are understood based on processes designed at THC.

Capital cost components that vary between alternatives have been applied to the capital cost of each alternative. Alternative 1 was the baseline for this comparison, so the change in cost is in comparison to Alternative 1 costs. This comparison is presented in Table 3-10. Costs shown in parenthesis indicate a savings would be achieved.

Table 3-10 Primary Treatment Alternative Capital Cost Comparison

EFFECTED PROCESSES[1]	ALTERNATIVE 1	ALTERNATIVE 2	ALTERNATIVE 3
Secondary Treatment (Basins, Blowers, Clarifiers, Pumping)	-	-	($8,919,000)
Sludge Thickening (Gravity Thickening, DAFs, WASSTRIP)	-	-	$1,950,000
Digestion, Dewatering, Sidestream	-	-	$1,752 ,000
Total Capital Cost Delta			**($5,217,000)**
Adjusted Estimated Capital Cost	**$10,438,000**	**$7,685,000**	**$6,184,000**

- Capital cost differences associated with Primary Treatment and Pumping are not included here because the difference is already included in the capital costs shown in Table 3-8.

Table 3-10 indicates that there is a net capital cost benefit of approximately $5,217,000 by using disk filters. All the savings comes from benefits associated with secondary treatment. Since the disk filters have higher TSS and BOD removal rates, there is less load to the aeration basins. This results in an oxic zone volume reduction of approximately 400,000 cubic feet. Less load to the basin also means there is less airflow required.

Additional secondary treatment capital cost savings is associated with the secondary clarifiers and the secondary clarifier sludge pump station. For the purposes of this primary capital cost evaluation, it was assumed the existing secondary clarifiers are reused. This decision will be based on hydraulic and site layouts that are beyond the scope of this TM and will be discussed in future TMs. When reusing the existing secondary clarifiers, Alternatives 1 and 2 require an additional three similarly-sized secondary clarifiers. Alternative 3 requires two additional similarly sized secondary clarifiers, so there is a savings associated with one less clarifier. If there is one less secondary clarifier, then there is a reduction in required secondary sludge pumping, resulting in savings at the secondary pump station. The Secondary Treatment design is detailed in TM 3.

Overall, the required sludge thickening associated with disk filters results in an additional cost of nearly $2 million. All this additional cost is due to providing two additional gravity thickeners. Since the filter backwash solids are estimated to be around 0.2 percent solids, there is an additional thickening step required to get to the typical primary sludge thickness. In addition to the two gravity thickeners, this cost also includes additional pumps that are associated with these thickeners. The overall sludge thickening cost associated with using disk filters would be higher if not for a cost savings associated with smaller dissolved air flotation thickeners (DAFs) and a smaller phosphorus recovery system. Since disk filters have a higher solids removal rate than traditional primary clarifiers, there are fewer secondary solids. This reduction in secondary solids allows for each DAF to be a little smaller in diameter, and the WASSTRIP phosphorus recovery system to be smaller. The savings associated with fewer secondary solids is not enough to mitigate the overall sludge thickening cost of adding gravity thickeners.

Table 3-10 indicates that using disk filters for primary treatment results in an increase in capital cost due to digestion, dewatering, and sidestream treatment. Of that cost adder, the majority is associated with an increase in diameter of each of the digesters. In total, there are three mesophilic anaerobic primary digesters and one secondary digester. When disk filters are used for primary treatment, approximately 10 percent more solids are expected to the digesters. It is estimated that each digester would increase in diameter accordingly. The capital cost of the dewatering process is not expected to change based on the primary treatment alternative. It is expected that the O&M cost would be affected based on increased weekly operation. Lastly, since approximately 10 percent more solids go through the dewatering process, the centrate equalization basin needs to increase in volume by 10 percent, resulting in a slight cost addition. All Biosolids Treatment design is detailed in TM 6.

The net benefit of $5,217,000 associated with using disk filters for primary treatment is subtracted from the original capital cost that was presented in Table 3-8. As shown in Table 3-10, there is no expected difference between downstream treatment processes in Alternative 1 and 2 (as explained in Section 3.2.4).

In addition, due to the affects the primary treatment technology selection has on the downstream treatment processes, operation and maintenance components that vary between alternatives have been added to the base operation and maintenance costs presented in Table 3-9. Like the capital costs, Alternative 1 was the baseline for these costs. This O&M comparison is shown in Table 3-11. Costs shown in parenthesis indicate a savings would be achieved.

Table 3-11 Primary Treatment Alternative O&M Cost Comparison

AFFECTED PROCESSES[1]	ALTERNATIVE 1	ALTERNATIVE 2	ALTERNATIVE 3
Secondary Treatment (Basins, Blowers, Clarifiers, Pumping)	-	-	($55,000)
Biosolids	-	-	$58,000
Total Comparison O&M Cost	-	-	$3,000
Adjusted Estimated O&M Cost	**$85,000**	**$96,000**	**$281,000**

- Primary Treatment differences are covered in the baseline O&M costs developed in Table 3-9

The Alternative 3 Secondary Treatment process has a net benefit due to power savings by determining the actual air demand. This resulted in having one less duty blower, which reduces the amount of equipment maintenance that is required. The Alternative 3 annual O&M costs due to Biosolids treatment is expected to add costs. Most of this cost is associated with an increase in the number of gravity thickeners and the pumps associated with the gravity thickeners. Having more equipment results in more power, labor, and equipment maintenance.

Overall, there is a $3,000 cost adder to the downstream treatment processes annual O&M when disk filters are used for Primary Treatment. It should be noted that Alternative 3 would result in an approximately 40 percent increase in digester gas produced at annual average conditions when compared to Alternatives 1 and 2. Currently, the savings associated with this increased gas is not accounted for in the O&M cost comparison. The increase in gas production is not added to this comparison because this depends on how the gas is used and the uncertainty of the renewable gas market conditions. An evaluation of the compressed natural gas (CNG) versus combined heat and power (CHP) is outside the scope of this TM; however, it should be noted that — in the future, once this project is closer to construction — a clearer picture of market conditions can be used to evaluate how best to use this excess gas.

3.3.4 Present Value

The 20-year present value (PV) calculations for each of the primary treatment alternatives are presented in Table 3-12. All PV estimates are based on the following assumptions:

- Cost year basis: 2020
- Nominal Discount Rate: 3.10 percent
- Inflation Rate: 1.90 percent
- Resulting Net Discount Rate: 1.20 percent

To calculate the total O&M cost over the 20-year life cycle, the annual O&M cost for each year is calculated by multiplying the previous year's annual O&M cost by the inflation rate. That annual O&M cost for that specific year is then corrected back to 2020 dollars, and the nominal discount rate is applied. The sum of all the annual present values is the overall present value O&M cost over 20-years.

Table 3-12 Capital, O&M, and NPV Cost Estimates (2020 $'s)

DESCRIPTION	CAPITAL COST	ANNUAL O&M COST	O&M PV (20 YEARS)	TOTAL PV
Alternative 1 – Traditional Clarification	$10,438,000	$85,000	$1,507,000	$11,945,000
Alternative 2 – Traditional w/ CEPT	$7,685,000	$96,000	$1,702,000	$9,387,000
Alternative 3 – Cloth Disk Filters	$6,184,000	$281,000	$4,981,000	$11,165,000

Alternative 1 had the highest capital cost and the lowest annual O&M cost. After evaluating these costs over a 20-year life cycle, Alternative 1 has the highest PV. Although Alternative 3 had the lowest capital cost after deducting the cost savings associated with the downstream treatment affects; however, the annual O&M costs associated with cloth disk filters results in a significant increase in the PV. Alternative 3 finishes in second place when comparing PV. Alternative 2 has the lowest PV by approximately $1,800,000 when compared to the next closest alternative.

It is important to note that salvage values have not been included in this PV evaluation. There would likely be a salvage value for concrete in the 20-year PV life cycle for each of these alternatives, but the similarity between each of these structures would result in an across-the-board increase of similar magnitude for all alternatives. Since the goal of the total PV is to differentiate alternatives so one can be selected, salvage value has not been included.

3.3.5 Triple Bottom Line Analysis

The Primary Treatment alternatives were compared through Triple Bottom Line (TBL) analysis. By factoring social and environmental considerations into the analysis along with economic information expressed as NPV, a more thorough comparison of alternatives can be achieved. The benefit score was then combined with the NPV to determine the benefit-cost of each alternative. The TBL criteria below in Table 3-13were approved by JCW in a biweekly progress meeting and are specific to MCR.

Table 3-13 Evaluation Criteria and Descriptions

CRITERIA	DESCRIPTION
Flexibility / Turndown	Is alternative flexible enough to successfully adjust to changing conditions (i.e. flow and load)? How much can be treated through the process?
Performance Reliability	Are there adjustable controls, process options, and/or equipment features available for operators to respond to an upset? Is alternative resistant to an upset, and what are the consequences if an upset does occur? Is alternative a proven technology?
Operational Complexity / Maintenance	How complex is the alternative to operate, control, and maintain? Does the alternative rely on more system components operating together? Are there major scheduled replacements and cleanings?
Layout / Constructability	How easily and cost-effectively can the alternative be phased to meet the start-up and construction constraints? How well does the alternative fit on the site? Do the facilities lay out in an orderly fashion (e.g., do trucks have to drive to through several facilities to access their final destination)?
Social Impacts	How well does the alternative prevent off-site impacts to public perception such as truck traffic, noise, odor, visual aesthetics, etc. and can these impacts be easily mitigated? (Impacts from construction activities are excluded.)
Environmental Impacts	How well does the alternative minimize impact to environment in terms of carbon footprint (during construction and use), ecosystem quality, and resource use?
Safety	How well does the alternative minimize safety risks to the plant staff and the public, and can the risks be mitigated?
Ease of Regulatory Acceptance	How difficult will alternative be to obtain EPA and KDHE regulatory acceptance? Could alternative acceptance be achieved in desired schedule?

Table 3-14 is a summary of the weighted scores for the Primary Treatment Alternatives. A ranking of five (5) means either this is the most important or most positive impact. A ranking of one (1) means either the is the least important or most negative impact.

Table 3-14 Primary Treatment Alternatives Triple Bottom Line Scoring

CRITERIA	RELATIVE WEIGHT	ALTERNATIVE 1 – TRADITIONAL CLARIFIERS		ALTERNATIVE 2 – TRADITIONAL W/ CEPT		ALTERNATIVE 3 – CLOTH DISK FILTERS	
		RANKING	WEIGHTED SCORE	RANKING	WEIGHTED SCORE	RANKING	WEIGHTED SCORE
Flexibility / Turndown	15%	3	4.5	3	4.5	4	6
Performance Reliability	20%	3	6	4	8	5	10
Operational Complexity / Maintenance	20%	5	10	4	8	3	6
Layout / Constructability	10%	3	3	3	3	3	3
Social Impacts	10%	3	3	3	3	3	3
Environmental Impacts	10%	3	3	2	2	3	3
Safety	10%	4	4	3	3	4	4
Ease of Regulatory Acceptance	5%	5	2.5	4	2	3	1.5
Total Weighted Score	100%	36		33.5		36.5	
Note: Rankings: 5 = Most Important or most positive impact. 1 = Least Important or most negative impact.							

To generally summarize the results of the TBL scoring, Alternative 1 has a proven track record that JCW has familiarity in operating. It also has the largest footprint of all the alternatives. Alternative 2 is similar to Alternative 1, with the only exception being increased concerns with chemicals at peak flows. There are safety concerns associated with handling chemicals, as well as the operational concerns of relying on chemical equipment to startup at peak flows when it has not been operating on a continual basis. The safety and operational concerns associated with chemicals result in a reduced score for environmental impacts and safety. Alternative 3 does not have many existing installations, which impacts the ease of regulatory acceptance. The other criterium where disk filters are negatively impacted is operational complexity and maintenance. Alternative 3 scores lowest here because the filter media needs to be replaced every few years. The benefits of disk filters are flexibility, turndown reliability, and performance reliability. Flexibility and turndown are a function of six filters that are relatively easy to bring online. In comparison, Alternative 1 has four units and Alternative 2 has three units. It is also more difficult to bring a clarifier offline. Performance of disk filters is consistent no matter what the influent flow is since treatment is a function of filtration instead of settling (as is the case with a clarifier).

3.3.6 Cost/Benefit Scoring

The sum of the TBL scoring can be converted to the normalized benefit score based upon the highest scoring alternative. The benefit scores for each alternative is then divided into the respective NPV to express the benefit score in economic terms. Table 3-15 contains the NPV to the normalized benefit ratio for Alternatives 1, 2, and 3.

Table 3-15 Primary Treatment Alternatives NPV / Normalized Benefit Ratio

CRITERIA	ALTERNATIVE 1 – TRADITIONAL CLARIFIERS	ALTERNATIVE 2 – TRADITIONAL WITH CEPT	ALTERNATIVE 3 – CLOTH DISK FILTERS
Total Weighted Score	36	33.5	36.5
Normalized Benefit Score	0.99	0.92	1.0
NPV Cost	$11,945,000	$9,387,000	$11,165,000
NPV/ Normalized Benefit Ratio	**12,066,000**	**10,203,000**	**11,165,000**

4.0 Summary of Findings and Recommendations

4.1 PRELIMINARY TREATMENT

It is recommended that the fine screening and grit removal facilities are housed together in a new Headworks Building. The fine screening system should include three channels, each with a shallow flow through perforated plate fine screen with one-fourth-inch openings. Screenings will be directed to a dumpster via a sluice trough and two washer-compactors. Grit removal should include two free vortex Headcell units. In addition, the grit removal system should be provided with two washer-dewatering units, and two slurry grit pumps. This preliminary treatment system is similar in operation and size to JCW's Tomahawk Creek WWTF, resulting in an increased cost certainty for the preliminary treatment system.

The site location and elevation of the Headworks Building will be determined in TM 8 - Site Optimization and Maintenance of Plant Operations (MOPO). Influent pumping improvements will also be determined with this TM.

4.2 PRIMARY TREATMENT

Although the lowest NPV / Normalized Benefit Ratio was Alternative 2, the recommended Primary Treatment Alternative is traditional clarifiers. Traditional primary clarifiers are the most conservative alternative on a cost and footprint basis. On a high-level facility plan that will be built several years in the future, it is appropriate to have some conservatism in the recommendation.

When the MCR WWTP Expansion occurs in the future, an update to the primary treatment evaluation is recommended. The biggest potential future changes to Alternative 2 would be fluctuations in the chemical market costs. In addition, Alternative 3 could have some technology advancements that make it more appealing. Based on the current primary treatment evaluation, a potential value engineering move could be to switch to traditional clarifiers with CEPT when the actual expansion is in design if the traditional clarifier option is carried forward in the facility plan. This would result in minimal design changes and have a significant cost savings.

Alternative 2 – Traditional Clarifiers with added CEPT at 38 mgd has the best NPV / Normalized Benefit Ratio. The biggest benefit for this alternative was one less clarifier than Alternative 1. The reduced capital cost associated with one less unit was a significant benefit. The O&M costs associated with chemical handling and the TBL scoring reductions for chemical handling were not enough of an impact to offset the benefit of one less unit.

Although Alternative 3 was not the recommended alternative, using cloth disk filters for primary treatment does have several benefits. As indicated in Table 3-8, disk filters had the lowest capital cost, due to the secondary treatment process benefits. The increased removal rate of disk filters has many benefits, including a smaller aeration basin, a reduction in blower units, reduced WAS processing and handling, and increased digester gas production. In addition, when comparing all primary treatment alternatives, disk filters have the smallest footprint by a substantial margin even though the Mill Creek Regional WWTP site is not limited for space. This was reflected in the TBL scoring criteria. For sites that are space limited, the benefit of disk filters becomes even greater.

Currently, there are three main disadvantages with using cloth disk filters in a primary treatment application. The first disadvantage is the high volume of backwash water produced. Directing all backwash flow to the gravity thickener/fermenter would result in significantly higher overflow rates and a dilute VFA concentration. These effects would be detrimental to RAS fermenter performance, which depends on little turbulence and high VFA concentrations to create conditions suitable for phosphorus release; therefore, the backwash water requires an intermediate thickening step where the thickened backwash solids are sent to the gravity thickener/fermenter. For this costing effort, the backwash thickening technology was selected as gravity thickeners for their ease of operation. Technologies such as belt filter presses and rotary drum thickeners are also viable alternatives. Thickening of backwash solids is an ongoing research topic for Aqua-Aerobics. If disk filters for primary treatment is to become more common, it is expected that this design aspect will be optimized as experience is gained.

The second disadvantage with using cloth disk filters for primary treatment is the associated annual O&M cost. In this evaluation, Alternative 3 had the lowest capital cost and the best TBL scoring. The reason it finished second was due to the annual O&M costs. Over a 20-year life cycle, these O&M costs impact the total NPV. The bulk of the O&M cost is due to labor and equipment maintenance. The labor piece of this cost is hard to mitigate because this alternative has the most equipment. The equipment maintenance piece of this cost could potentially be improved with improved filter media and/or more primary treatment installations. Currently, the manufacturer recommends replacing the filter disks approximately every seven years in a tertiary treatment application. It is assumed in a primary treatment application that this replacement frequency would be accelerated, but with few installations it is hard to accurately predict what the filter life is. Additionally, if somehow the filter media was improved to have 10 years of life, that would incentivize the use of disk filters.

The third disadvantage is the lack of installations of disk filters for primary treatment. A lack of installations leads to a lack of real world data and potential troubleshooting. As a technology becomes more common, the understanding of typical issues becomes more known. There is a possibility this increases in the future, and, if desired, a pilot using disk filters for primary treatment could be evaluated.

Based on conclusions in this TM, traditional primary clarifiers will be carried forward as the primary treatment technology. This recommendation will be the assumption in future TMs. The design criteria for the Primary Clarifiers will be as shown in Table 3-1. The site location and elevation of the primary clarifiers will be determined during the development of future TMs.

DRAFT

MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 3
Secondary and Sidestream Treatment

JCW NO. MCR1-BV-17-12
BV PROJECT 403165

PREPARED FOR

JOHNSON COUNTY KANSAS
Wastewater

OCTOBER 9, 2020

BLACK & VEATCH

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Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CANDO	Coupled Aerobic-anoxic Nitrous Decomposition Operation
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEA	Cost Effective Analyses
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
CIPP	Cured-in-place Pipe
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSBR	Continuous Sequencing Batch Reactor
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
D	
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry-weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered

Abbreviation	Meaning
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand
ffCOD	Flocculated Filtered Chemical Oxygen Demand
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
ft	Feet
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
H	
HB	Hallbrook Facility
HDD	Horizontal Directional Drilling
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
Hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration
IC	Internal Combustion
IFAS	Integrated Fixed-Film Activated Sludge

Abbreviation	Meaning
in	Inches
IND	Industrial
INF	Influent
IP	Intellectual Property
IPS	Influent Pump Station
IR	Irrigation Use
IRR	Irrigation
IW	Industrial Water Supply Use
J	
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDHE	Kansas Department of Health and Environment
K_e	Light Extinction Coefficient
kWh	Kilowatt-hour
L	
L	Length, Liter
lb	Pound
LF	Linear Feet
LOMR	Letter of Map Revision
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter
mgd	Million Gallons per Day
µg/L	Micrograms per Liter

Abbreviation	Meaning
min	Minute, Minimum
mJ	Millijoules
MLE	Modified Ludzack Ettinger
MLSS	Mixed Liquor Suspended Solids
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow
mmBtu	Million British Thermal Units
MOPO	Maintenance of Plant Operations
mpg	Miles per Gallon
MPN	Most Probable Number
N	
N	Nominal Thrust
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NFIP	National Flood Insurance Program
NH3-N	Total Ammonia
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
PV	Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow
PE	Primary Effluent
PFE	Primary Filtered Effluent
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow
PIF	Peak Instantaneous Flow
PLC	Programmable Logic Controller
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day
pph	Pounds per Hour
PPI	Producer Price Index
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet-weather Flow
Q	
Q	Flow
R	
RAS	Return Activated Sludge
RAS	
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
Sec	Second, Secondary
SF	Square Foot
SG	Specific Gravity

Abbreviation	Meaning
SLR	Solids Loading Rate
SMP	Stormwater Management Program, Shawnee Mission Park Pump Station
SND	Simultaneous Nitrification/ Denitrification
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station
SRT	Sludge Retention Time
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
TYP	Typical

Abbreviation	Meaning
U	
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VFA	Volatile Fatty Acids
VFAs	
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	Width
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation –Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WLWater LevelWK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

The purpose of this technical memorandum (TM) is to summarize the conceptual design of the secondary and sidestream deammonification facilities at the Mill Creek Regional (MCR) Wastewater Treatment Plant (WWTP). This TM includes a discussion of available treatment technologies, design criteria of the selected technology, footprint and layouts of the selected technology, capital costs, and operational and maintenance (O&M) costs.

This TM is one in a series of technical memoranda that will be incorporated into a Facility Plan report summarizing a future expansion of the MCR plant. Additional treatment processes and site optimization of these treatment facilities will be outlined in future TMs.

1.1 BACKGROUND

Prior to this Facility Plan for MCR, an extensive alternative analysis was done for the Tomahawk Creek (THC) WWTP Expansion. The results of this analysis can be used to inform the planning of the MCR Expansion. The THC WWTP is a good comparison because it is a similarly-sized facility (19 million gallons per day (mgd) annual average (AA) flow), with similar wastewater characteristics, is owned and operated by JCW, and has actual market costs for treatment technologies provided by a Contractor.

In August of 2014, Johnson County Wastewater (JCW) retained Black & Veatch (BV) for the project definition phase of the Tomahawk Creek (THC) WWTP Expansion. The primary objective of the project definition phase was to confirm, through alternative development and evaluation, the optimal and proven treatment strategies throughout the WWTP for nutrient removal to meet current and anticipated future NPDES limits for design flows. Evaluation of these alternatives consisted of utilizing the JCW's Triple Bottom Line (TBL) approach to evaluate non-economic factors in addition to developing capital and operating costs for each alternative. Each treatment process evaluation was presented to JCW to select a recommended technology to be carried forward through design and construction.

After the project definition phase, the THC WWTP Expansion was continued into detailed design, followed by construction. The construction is scheduled to be completed in 2021. During the detailed design phase, some of the selected treatment technologies were re-evaluated and eventually revised as part of a value engineering effort. The treatment technologies that were part of the final design and eventually carried into construction serve as a valuable comparison for the MCR WWTP.

From TM 1 – Background, Flows, Loadings, and NPDES Limits, the design flows for the WWTP were established (as shown in Table 1-1). It should be noted that the secondary process will treat up to the peak secondary flow of 63 mgd. Flows exceeding 63 mgd will receive auxiliary treatment as described in TM 4 – Auxiliary Wet Weather Treatment.

The sidestream deammonification process will treat all dewatering centrate/filtrate flows created in the biosolids treatment process. This TM includes the design of both the secondary and sidestream treatment processes because the sidestream deammonification process is an important consideration in the design of the secondary treatment process.

Table 1-1 MCR Design Flows

	DIURNAL LOW AA STARTUP	AA STARTUP	AA ULTIMATE	MAX MONTH	PEAK SECONDARY	PEAK DAY
MCR Design Flows	6.0[1]	12.0	21.0	31.5	63.0[2]	126.0
[1] Historically this is 1/2 of the diurnal high (AA startup). [2] Peak secondary capacity is 3 times AA Ultimate (3Q).						

1.2 SECONDARY TREATMENT (BNR)

1.2.1 Summary of Available Technologies

Rather than starting a completely new and independent evaluation of available technologies for secondary and sidestream treatment at the MCR WWTP, it was decided to build off of the evaluation completed for the THC WWTP Expansion (which is currently under construction). In selecting secondary treatment alternatives for evaluation at the THC WWTP, a matrix was developed to summarize and screen 17 technologies against initial criteria. This narrowed the field to four alternatives for further consideration. The driving criteria used to narrow the field of technologies were: 1) a small footprint to minimize Tomahawk Creek flood impacts, and 2) the ability to treat to low effluent ammonia and phosphorus concentrations.

Biological phosphorus removal was selected over chemical phosphorus removal to reduce operational chemical costs. Enhanced biological phosphorus removal (EBPR) and activated granular sludge (AGS) were evaluated as options to support biological phosphorus removal. Specifically, Black & Veatch's latest BNR configuration — sidestream EBPR (S2EBPR) in a plug flow configuration — was recommended due to its ability to support stable and robust EBPR performance despite seasonal and influent quality variations that affect conventional EBPR (See Section 1.2.2). S2EBPR may be configured and integrated with other technologies for process intensification, such as integrated fixed film activated sludge (IFAS), membrane bioreactors (MBRs), and ballasted activated sludge (e.g., Evoqua BioMag®).

The four technologies recommended for in-depth evaluation were as follows:

- Alternative 1 – S2EBPR Plug Flow with Integrated Fixed Film Activated Sludge
- Alternative 2 – S2EBPR Plug Flow with Membrane Bioreactors
- Alternative 3 – S2EBPR Plug Flow with BioMag®
- Alternative 4 – Activated Granular Sludge (AGS)

Through discussions, workshops, and economic and non-economic analysis, Alternative 1 – IFAS was initially selected as the secondary treatment technology to implement at the THC WWTP; however, a significant value engineering effort was undertaken in detailed design and the excess flow holding basin was removed from the project, eliminating the need for small footprint technology. The design team took advantage of an opportunity to reduce the cost of secondary treatment by expanding the basin's Oxic Zone for a conventional activated sludge process and eliminating IFAS.

For the MCR WWTP, a decision was made to pursue biological phosphorus removal over chemical phosphorus removal. Chemical phosphorus removal was eliminated from consideration due to associated high operational costs at plants similar in size to MCR WWTP. As shown in Figure 1-1, the secondary options considered for the MCR WWTP included S2EBPR in a plug flow configuration, S2EBPR in an oxidation ditch configuration, and AGS. Oxidation ditches are typically implemented at facilities smaller than the MCR WWTP with no footprint constraints and minimal staffing. The JCW staff is trained and comfortable operating the plug flow activated sludge processes, which provide footprint advantages, energy saving advantages, and possible benefits to digester gas yield. AGS is a viable option, but has limited installations in North America at the time of this evaluation. If interest exists, a pilot is recommended to assess its potential performance at the MCR WWTP; therefore, S2EBPR in a Plug Flow configuration was selected for MCR.

Four configurations of the plug flow S2EBPR were considered: 1) IFAS, 2) membrane aerated biofilm reactors (MABRs), 3) MBRs, and 4) conventional activated sludge. Because the MCR WWTP site is large, supporting the conventional activated sludge footprint is not a concern. This eliminates the need for process intensification by IFAS or MBRs, which prove themselves desirable in restricted footprint applications. The MABR is a fairly new technology to the market. Only one full-scale North America installation was active at the time of evaluation. The MABR provides energy efficiency in addition to process intensification, but is not cost competitive if based on energy efficiency alone. As seen in Figure 1-1, S2EBPR Plug Flow with conventional activated sludge was selected. Implementing the same biological nutrient removal (BNR) process at the MCR WWTP that is being applied at the THC WWTP will increase commonality between JCW treatment facilities and reduce operation and maintenance costs from a complete utility perspective.

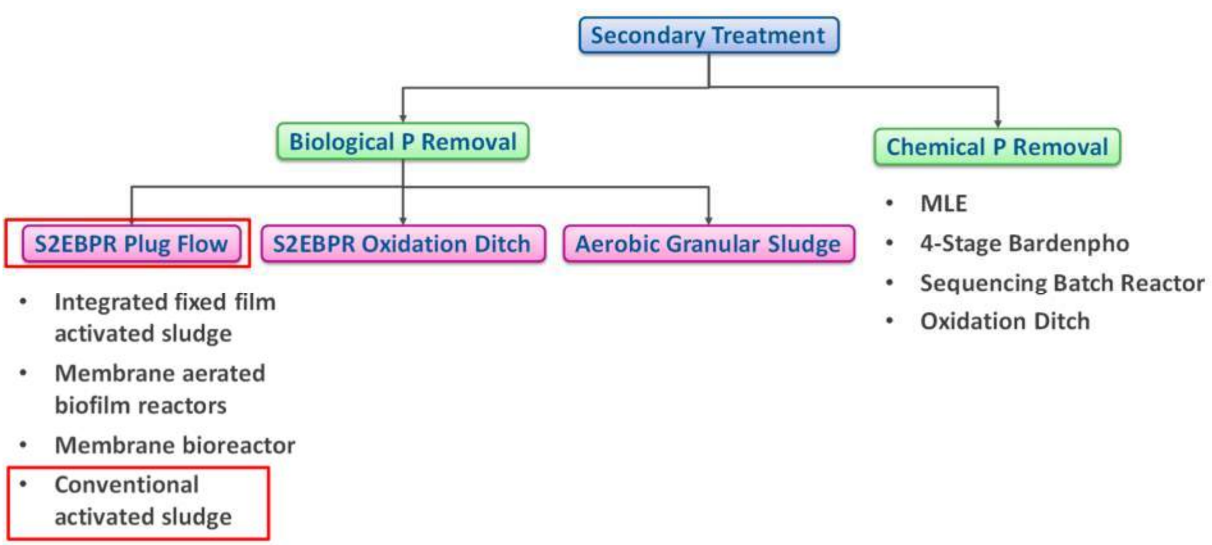


Figure 1-1 Secondary Treatment Alternatives

1.2.2 Plug Flow S2EBPR

The new BNR facility will utilize a plug flow S2EBPR configuration. Local treatment facilities currently using this process include Cedar Creek WWTP (Olathe, KS) and the Tomahawk Creek WWTP (Johnson County Wastewater). The S2EBPR configuration consists of seven separate treatment zones and are described as follows:

Pre-Anoxic Zone (PAX) – The primary function of the pre-anoxic zone is to remove nitrate and dissolved oxygen (DO) from the return activated sludge (RAS). This helps protect the anaerobic zone from nitrate and oxygen consuming volatile fatty acids (VFAs).

Anaerobic Zone / RAS Fermenter (AN) – The anaerobic zone serves three major process functions. The first function is to condition the phosphorus accumulating organisms (PAOs) for phosphorus release. The second function is fermentation to produce VFAs from the incoming readily biodegradable chemical oxygen demand (rbCOD). The last function is sludge conditioning for filament control. Carbon sources (gravity thickener/fermenter overflow and supplemental carbon) are added to help drive these process functions.

First Anoxic Zone (AX) – The first anoxic zone receives forward flow from the anaerobic zone, RAS and return flow from the end of the oxic zone. This return flow transports nitrates to the anoxic zone for removal using denitrification to convert the nitrates to nitrogen gas.

Oxic Zone (OX) – This zone provides an aerobic environment for carbonaceous BOD removal, nitrification, and phosphorus uptake. The oxic conditions produced in the first oxic stage is needed to trigger the "luxury uptake" of phosphorus by PAOs.

Second Anoxic Zone (AX) – The second anoxic zone is needed to remove additional nitrate from endogenous oxygen demand from the mixed liquor suspended solids (MLSS). Supplemental carbon is dosed in this location to aid in nitrate removal as needed.

Second Oxic Zone (OX) – The second oxic zone is needed to add DO into the BNR effluent ahead of the final clarifiers, which prevents secondary release of phosphorus and denitrification in the clarifiers.

Swing Zone (SW) – The swing zone is located between the second anoxic and second oxic zones and can serve as additional volume for either zone based on process needs.

Figure 1-2 shows each treatment zone in sequence for one treatment train of an S2EBPR BNR basin. Similar to the THC WWTP, the BNR Basin constructed at MCR WWTP will consist of four parallel treatment trains. Figure 2-1 in Section 2-1 shows the preliminary layout of the MCR BNR Basin based on loading criteria.

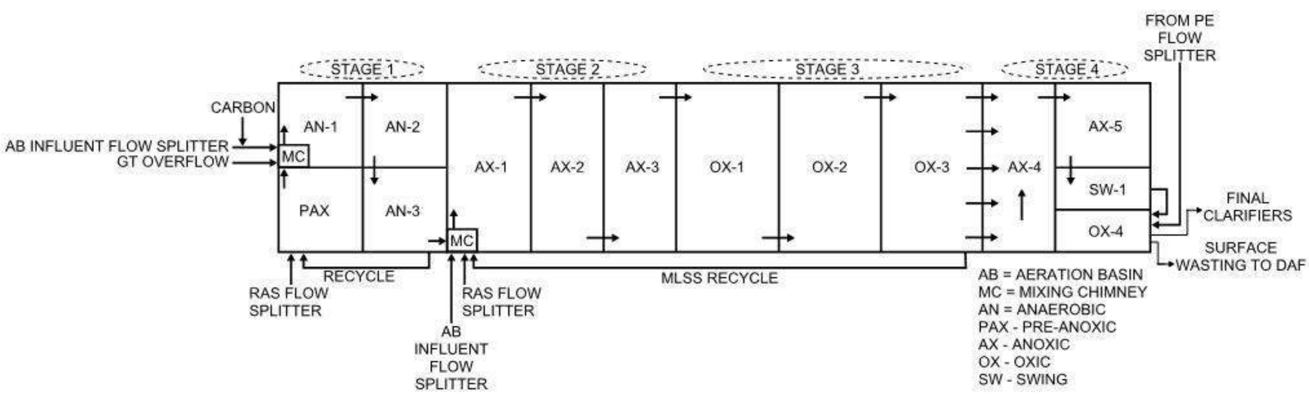


Figure 1-2 Plug Flow S2EBPR Schematic

1.3 AERATION BLOWERS

Two types of BNR basin blower technologies were evaluated for installation at the THC WWTP:

- High-speed gearless single-stage centrifugal (turbo)
- Integrally geared single-stage centrifugal (single-stage)

At the time of the THC WWTP evaluation, it had been reported that JCW had an installation with single-stage blowers manufactured by Siemens (under the Turblex name at the time) and service responsiveness had been a concern for JCW. Due to unfavorable history with Siemens and the relatively high associated capital cost at the design blower capacity in comparison to turbo blowers, single-stage blowers were eliminated from the blower evaluation. Because of the proven reliability of this technology when applied to plug flow BNR processes, and to increase commonality across JCW facilities, gearless turbo blowers are selected to for BNR aeration at MCR WWTF.

1.3.1 Gearless Turbo Blowers

Turbo blower technology utilizes a high-speed motor with a single-stage impeller mounted directly to an extended motor shaft. Non-contact bearings, either air foil or magnetic, eliminate the need for lubricating oil and mechanical wear of parts. The motor is driven by a high frequency adjustable frequency drive (AFD) and speed is used to control capacity. Depending on blower manufacturer and size, the maximum operating speed may range from 14,000 to 30,000 rpm or more. Six-pulse AFDs are provided by the manufacturer as an integral component of the blower package. Harmonic filters are recommended to mitigate harmonics in the plant power system from the high frequency drive. Some of the turbo blower manufacturers install the harmonic filter within their package and others require an external third party installed harmonic filter.

1.4 FINAL CLARIFIERS

The BV standard approach to final clarification is circular final clarifiers with spiral scrapers. Since this is consistent with JCW's standard approach to final clarification, and MCR currently has two existing final clarifiers, this is the recommended technology for selection when the expansion of MCR WWTP occurs. Similar to THC WWTP, the final clarifiers at MCR WWTP will be capable of being dosed with ferric chloride upstream of the clarifiers to aid in effluent phosphorus removal.

1.5 SIDESTREAM DEAMMONIFICATION

1.5.1 Summary of Available Technologies

An evaluation during the Project Definition Phase of the THC WWTP project concluded sidestream deammonification treatment for enhanced removal of ammonia and nitrogen aligned with project goals from a TBL standpoint. Capital costs associated with the inclusion of a sidestream deammonification process were calculated to be offset by operational savings over a seven-year period.

The following sidestream deammonification process alternatives were evaluated on selection criteria established in the TBL evaluation:

- Alternative 1 – WorldWaterWorks DEMON®
- Alternative 2 – Veolia (Kruger) Anita™ Mox
- Alternative 3 – Paques Anammox®

Suez Cleargreen™ was approached as a fourth alternative to be evaluated; however, Suez did not have a full-scale operating facility at the time of the evaluation and decided not to pursue the project.

Paques Annamox® was ruled out for implementation at THC WWTP due to the limited number of North American installations at the time of evaluation. For the THC WWTP, a detailed present worth evaluation between DEMON® and Anita™ Mox showed similar capital and O&M costs for the two systems. In their configurations at that time, the main differentiator was retention of biomass in the anammox reactor. DEMON® is a suspended growth process while Anita™ Mox grows the anammox bacteria on plastic carrier media which are retained in the tank by sieves. It was felt that the Anita™ Mox system had a significant benefit in that it is almost impossible to lose the biomass out of the reactor while DEMON® posed a much higher risk of accidental loss of biomass from the bioreactor using hydrocyclones. Since the anammox bacteria are very slow growing, loss of biomass could take a couple of months to replace, short of hauling in purchased seed from an offsite location. Because of the biomass loss issue, Anita™ Mox was selected for the THC WWTP. DEMON® has made some process improvements since that time, namely the addition of a fine screen on the effluent discharge to retain the larger anammox granules. Other than the addition of a screen with DEMON® there have been no significant changes to the either of these two basic sidestream processes. With the desire for commonality across JCW facilities, the Anita™ Mox process has been selected for installation at the MCR WWTP.

1.5.2 Anita™ Mox

The Anita™ Mox process by Veolia (Kruger) is a completely mixed, continuously aerated, flow through, moving bed bioreactor (MBBR) process. The Anita™ Mox process retains anammox bacteria by fostering biofilm growth on plastic carrier media, which are retained in the reactor by sieves. The process is controlled automatically using a PLC control scheme which incorporates pH and DO. Both micronutrients and caustic will be dosed as needed for process control. Once an Anita™ Mox system is successfully started up, feedback has shown plants are typically able to ween off of micronutrients and sodium hydroxide; however, results vary by plant. Figure 1-3 shows a schematic of the sidestream deammonification process.

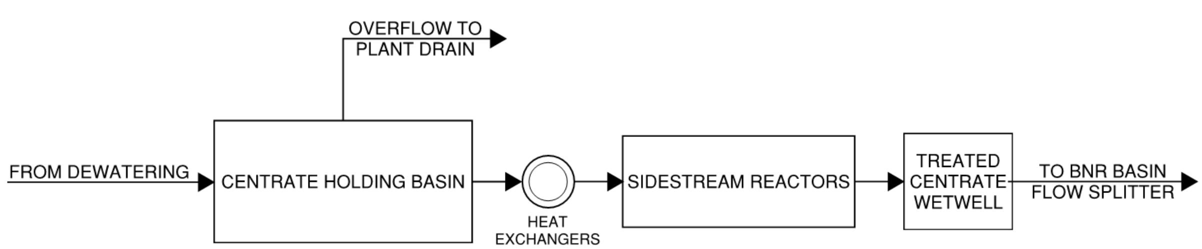


Figure 1-3 Sidestream Deammonification Treatment Schematic

2.0 Basis of Design Criteria

2.1 SECONDARY TREATMENT (BNR)

The secondary influent loading criteria that were developed for conventional primary treatment in TM2 were used as the basis to design the secondary treatment alternatives for this TM. The BNR Basin will be a large concrete structure divided into 4 trains, each with a 20-foot side water depth in the oxic zones. Refer to Figure 2-1 for the proposed BNR Basin layout. Table 2-1 and Table 2-2 summarize BNR influent loading and the total required volume of each treatment zone.

Table 2-1 BNR Influent Loads

PARAMETER	BNR BASIN INFLUENT LOADING, PPD [1]	
	Annual Average with 15-day SRT	Maximum Month Winter
COD	52,631	77,103
TSS	26,110	41,247
VSS	22,655	35,740
BOD	23,296	33,535
NH3-N	4,047	5,156
TKN	5,972	8,001
TN	6,263	8,373
OP	403	551
TP	725	1,052

[1] Includes Primary Effluent and Gravity Thickener Overflow Loads.

Table 2-2 BNR Basin Volume

PARAMETER	REQUIRED VOLUME, CF[1]
Pre-Anoxic Zone	61,500
Anaerobic Zone	168,100
Anoxic Zone	308,700
Oxic Zone	1,300,000
Second Anoxic Zone + Swing Zone	210,900
Second Oxic Zone	29,700
Total Basin Volume	2,078,900
[1] Total required volume for each zone type across all trains.	

The layout of the BNR Basin has been optimized to minimize large diameter piping and concrete required. To achieve this goal, each basin train is a mirror image of the adjacent train. This allows two basin trains to share a common pre-anoxic zone, anaerobic zone, and effluent box. During the site optimization associated with TM 8 – Site Optimization & MOPO, the basin will be oriented on site to further minimize large diameter piping.

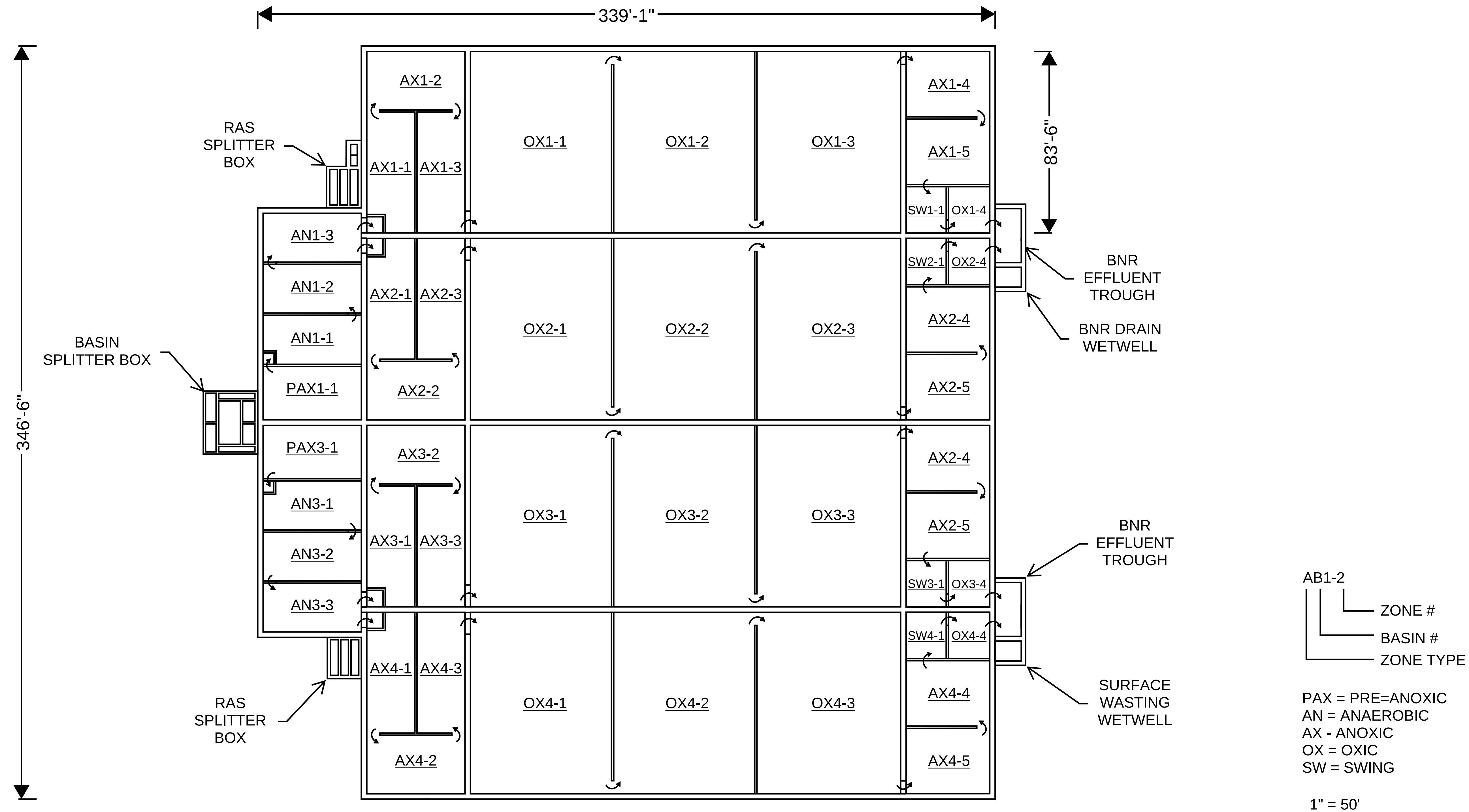
The 4 BNR trains in total will house 32 submersible mixers, 7 low capacity submersible pumps, and 4 submersible horizontal propeller pumps. Table 2-3 and Table 2-4 summarize mixer and pump design criteria. The oxic stage of each basin train is divided into three separate zones. The first two zones contain fine bubble diffusers to maximize oxygen transfer, while the third zone contains coarse bubble diffusers to better support cycling of air to avoid over-aerating the MLSS being recycled to the anoxic zone. The swing and second oxic zones of each train will contain coarse bubble diffusers.

Table 2-3 BNR Basin Mixer Design Criteria

PARAMETER	DESIGN CRITERIA			
Mixer Locations	Pre-Anoxic Zone	Anaerobic Zone	Anoxic Zone	Swing Zone
Mixer Quantity	2 (1 per pair of trains)	6 (3 per pair of trains)	20 (5 per train)	4 (1 per train)
Mixer Type	Submersible, Direct Drive			
Maximum Propeller Diameter, in	14.5			
Nominal Thrust, N	430			
Maximum Motor rating, Hp	2.5			

Table 2-4 BNR Basin Pump Design Criteria

PARAMETER	DESIGN CRITERIA				
Pump Application	BNR Anaerobic Feed	BNR Anaerobic Recycle	BNR Drain	BNR Surface Wasting	Mixed Liquor Recycle
Pump Location	BNR Splitter Box	North/South RAS Fermenter	East/West Drain Wetwell	Surface Wasting Wetwell	Oxic Zone 3
Pump Quantity	2	2	2	1	4
Pump Type	Low Capacity Submersible	Low Capacity Submersible	Low Capacity Submersible	Low Capacity Submersible	Submersible Horizontal Propeller
Installed Horsepower, Hp	5	5	12	3	30



JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 3 - SECONDARY AND SIDESTREAM TREATMENT
BNR BASIN LAYOUT
FIGURE 2 - 1

BLACK & VEATCH

2.2 BASIN BLOWER BUILDING

The high-speed gearless turbo blowers that will provide aeration to the BNR Basin will be housed in a single-story masonry structure consisting of at-grade blower and electrical rooms. The electrical room will be sized large enough to house electrical and control equipment for both the BNR Basin and Basin Blower Building. The building will be sized large enough to provide ample room for blower maintenance and removal via forklift. A roll-up door will be provided for blower removal. Refer to Figure 2-2 for the proposed Basin Blower Building layout.

2.2.1 High Speed Gearless Turbo Blowers

Table 2-5 summarizes the basin blower design criteria.

Table 2-5 BNR Basin Blower Design Criteria

PARAMETER	DESIGN CRITERIA
Quantity	5 (4 Duty, 1 Standby)
Type	Gearless Turbo
Rated Capacity, scfm	4,890
Turndown Capacity, scfm	2,590
Blower, hp (Each)	300
Rated Discharge Pressure, psig	10.6

A dedicated inlet filter / silencer housing will be provided for each blower that is piped to outside the blower room. Similarly, a silencer will be provided in each blowoff pipe to provide sound attenuation during periods of blowoff that occur during startup and shutdown of a unit. The discharge of each blower is piped to a common header outside of the Basin Blower Building. Flow from the discharge header is split into two sub-headers that are routed to the BNR Basin. Manual butterfly valves will be installed in the common discharge header to isolate the two BNR sub-headers to allow one half of the BNR process to operate if the aeration piping in the other half requires maintenance.

2.2.2 Supplemental Carbon System

Supplemental carbon is fed to the anaerobic and second anoxic zones in the BNR Basin to aid in nitrate removal. Feed to the anaerobic zones is designated to support the BNR process in the event of an upset condition. Feed to the second anoxic zone is also designated for an upset condition, but could additionally be used in the future to supplement the process for effluent nitrate control.

Potential means of supplying supplemental organic carbon to wastewater treatment include methanol, ethanol, MicroC©2000, acetic acid, sodium acetate, and glycerin. The chemical selected for supplemental carbon feed at THC WWTP (and subsequently MCR WWTP) was MicroC©2000 due to safety in handling compared to the other organic carbon sources considered. In addition, MicroC©2000 has greater flexibility to be used intermittently compared to the other organic carbon sources considered because of the wide range of heterotrophic organisms that can utilize it as a carbon source.

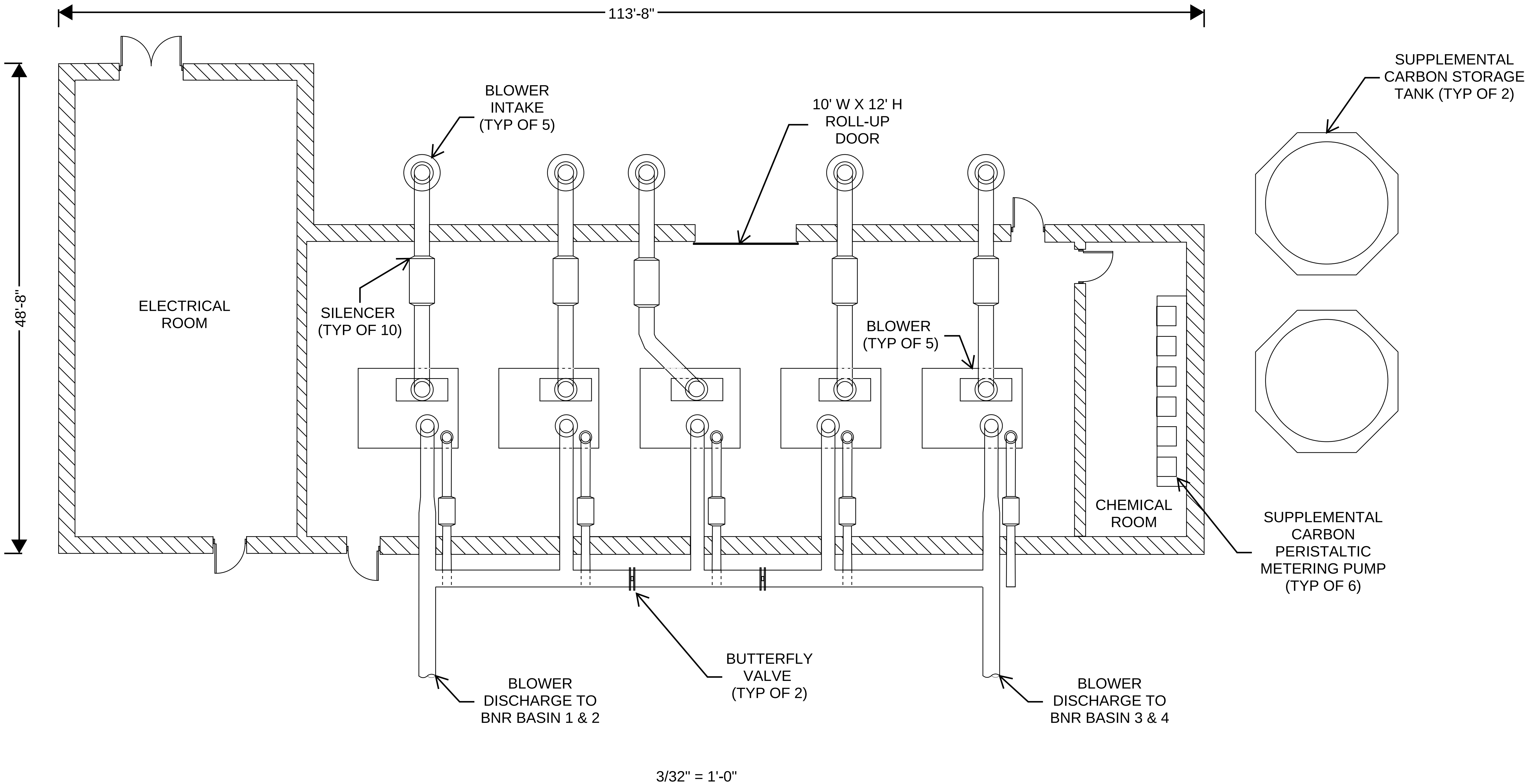
Because there are minimal safety requirements associated with the storage of MicroC©2000, and due to the proximity of the structure to the chemical application point, the supplemental carbon system will be housed within the Basin Blower Building. In order to limit exposure to chemicals, the supplemental carbon metering pumps will be located in a room adjacent to the blower room. Bulk storage of supplemental carbon will be located outside of the building in double contained polyethylene chemical storage tanks. Table 2-6 summarizes the design criteria for the supplemental carbon metering pumps.

Table 2-6 Supplemental Carbon Metering Pump Design Criteria

PARAMETER	DESIGN CRITERIA	
Pump Location	Basin Blower Building	Basin Blower Building
Pump Type	Peristaltic	Peristaltic
Pump Quantity	2	4
Application Point	BNR Basins 1/2 and 3/4 Anaerobic Zone 1	BNR Basins 1-4 Anoxic Zone 4
Flow Range, gph	0.30 – 30.0	0.15 – 15.0

2.2.3 BNR Basin Ferric Chloride System

To aid in effluent phosphorus removal in the event of an upset in the BNR process, ferric chloride may be dosed to polish phosphate through chemical precipitation. A standby ferric chloride system will be installed in the Digester Control Building, described in TM 6 – Biosolids Treatment. The system will consist of an 8,700-gallon fiberglass ferric chloride storage tank and two peristaltic metering pumps (duty/standby). In the event of a BNR upset, ferric will be pumped from the Digester Control Building to a designated feed point located upstream of the final clarifiers.



JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
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TM No. 3 - SECONDARY AND SIDESTREAM TREATMENT
BASIN BLOWER BUILDING LAYOUT
FIGURE 2 - 2

BLACK & VEATCH

2.3 FINAL CLARIFIERS

The existing Final Clarifiers 1 and 2 are located downstream from existing aeration basins (Completely-Mixed Cells 1 and 2). In order to meet future flow and loading requirements, additional clarifier capacity will be required at MCR WWTP. Two alternatives were identified to meet clarifier capacity requirements:

- FC Alternative 1 - Add three (3) 130-foot clarifiers and replace clarifier equipment in existing basins.
- FC Alternative 2 - Demolish existing clarifiers and construct four (4) 145-foot diameter clarifiers.

Downstream impacts between the two alternatives are limited to sludge pumping, as described in Section 2.4. If FC Alternative 1 is selected, the location of final clarifiers is fixed on site, whereas FC Alternative 2 provides the flexibility to move the location of final clarifiers around on site. The alternative selection will be made during the site optimization associated with TM 8 – Site Optimization & MOPO. Table 2-7 summarizes the final clarifier design criteria for each of the two alternatives.

Table 2-7 Final Clarifier Design Criteria

PARAMETER	FC ALT 1 – RETROFIT EXISTING CLARIFIERS	FC ALT 2 – INSTALL NEW CLARIFIERS
Quantity	5	4
Diameter, ft	130	145
SOR at 3Q, gpd/SF	950	1,190
SLR at 3Q, ppd/SF	27	27
Peak MLSS, mg/L	3,430	3,430

2.4 SLUDGE PUMPING

The existing Sludge Pumping Station is located near Final Clarifier 1 and 2. The pump station is equipped with three return activated sludge (RAS) pumps and four waste activated sludge (WAS) pumps. The RAS pumps currently take settled sludge from Final Clarifier 1 and 2 and pump it to the existing aeration basins Completely Mixed Cell 1 and 2. The WAS pumps currently discharge into Completely Mixed Cell 2.

If FC Alternative 1 is selected, the existing Sludge Pumping Station will continue to serve Final Clarifier 1 and 2 and a new pump station will be constructed nearby to serve the three newly-constructed final clarifiers. The existing Sludge Pumping Station was built with subsurface stub walls and foundation for expansion of the building in the event of future clarifier construction; however, expanding the existing pump station to accommodate three new clarifiers would create hydraulic and layout constrictions. For these purposes, it is not recommended that the existing pump station be expanded at the time of expansion.

A condition assessment of clarifier and sludge pumping equipment will need to be conducted to determine the viability of re-using existing equipment; however, it is likely that clarifier equipment and pumps will need to be replaced at the time of expansion of MCR WWTP.

Under FC Alternative 2, the existing sludge pump station and existing clarifiers will be demolished and new facilities will be constructed at an optimized location (as outlined in TM 8 – Site Optimization & MOPO).

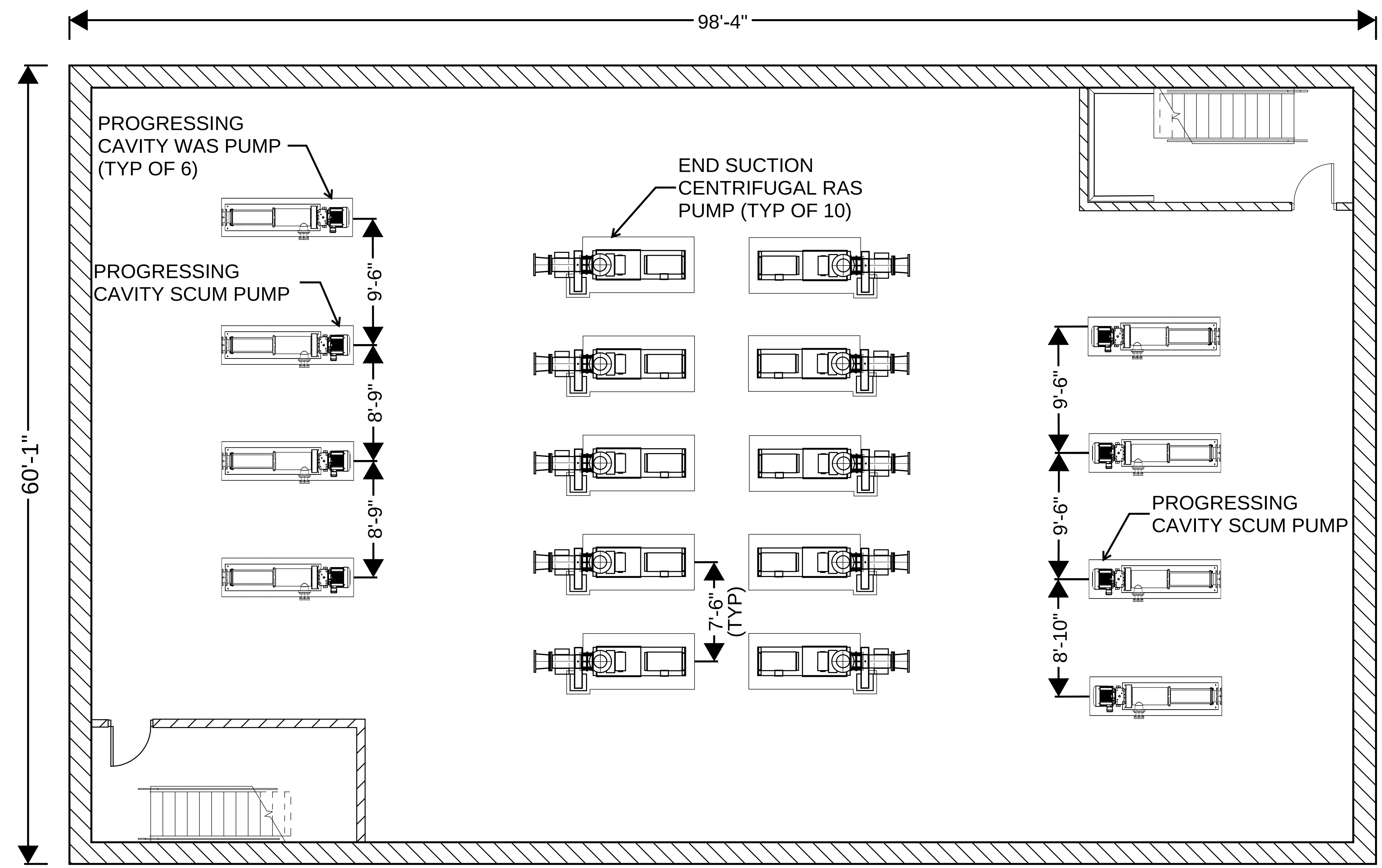
The new sludge pumping station shown in Figure 2-3 can be modified to service either final clarifier alternative. The pumping station will consist of a cast-in-place concrete pump room below grade and a masonry superstructure for storage and electrical equipment. Under FC Alternative 1, the below grade pump room will consist of 15 pumps. Two duty RAS pumps will be installed to service each clarifier along with two standby RAS pumps that can swing between clarifiers, totaling eight new pumps. RAS pump design criteria are summarized in Table 2-8. One duty WAS pump will serve each clarifier, along with a shared scum pump. Each header of WAS and scum pumps will have a standby swing pump to serve either function, totaling seven new progressing cavity pumps. WAS pump design criteria are summarized in Table 2-9. One quadrant of the pump room will be left vacant for pump maintenance and storage. Under FC Alternative 2, this quadrant would be occupied by two additional RAS pumps and one additional WAS pump placed in a similar arrangement to the other pumps in the basement.

Table 2-8 RAS Pump Design Criteria

PARAMETER	FC ALT 1 – RETROFIT EXISTING CLARIFIERS	FC ALT 2 – CONSTRUCT NEW CLARIFIERS
Number of Pumps	8 (6 Duty, 2 Standby)	10 (8 Duty, 2 Standby)
Type	Horizontal End Suction Centrifugal	
Pump Capacity, gpm	825	
RAS Return Rate	0.2Q – 1.25Q (4.2 mgd – 26.25 mgd)	
Motor Rating, hp	20	

Table 2-9 WAS Pump Design Criteria

PARAMETER	FC ALT 1 – RETROFIT EXISTING CLARIFIERS	FC ALT 2 – INSTALL NEW CLARIFIERS
Number of Pumps	7 (3 Duty WAS, 2 Duty Scum, 2 Standby)	8 (4 Duty WAS, 2 Duty Scum, 2 Standby)
Type	Progressing Cavity	
Pump Capacity, gpm	90	
Motor Rating, hp	15	



LOWER LEVEL

3/16" = 2'-0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 3 - SECONDARY AND SIDESTREAM TREATMENT
FINAL SLUDGE PUMP STATION LAYOUT
FIGURE 2 - 3

2.5 SIDESTREAM DEAMMONIFICATION

2.5.1 Sidestream Deammonification Building

The Sidestream Deammonification Building will be a slab on grade masonry structure located adjacent to the Centrate Equalization Basin, described in TM 6 – Biosolids Treatment. As shown in Figure 2-4, a mechanical room will house the boiler pumps, boilers, heat exchangers, and other non-pictured components of the Anita™ Mox process. An electrical room and two separate chemical rooms will be located on either side of the mechanical room.

Each chemical room will have only one entrance, located on the building exterior. The Micronutrient Room will house a two-pump skid along with a day tank resting on a dosing scale. Micronutrients used in the Anita™ Mox process will be delivered to the site in drums and transferred by staff into the micronutrients day tank. Adjacent to the Micronutrient Room is the Sodium Hydroxide Room, which will house a two-pump skid fed by the double contained carbon steel sodium hydroxide bulk storage tank located outside of the structure. To minimize heat loss from the deammonification process, the Anita™ Mox reactors themselves will be housed underground adjacent to the Centrate Equalization Basin, discussed in TM 6 – Biosolids Treatment.

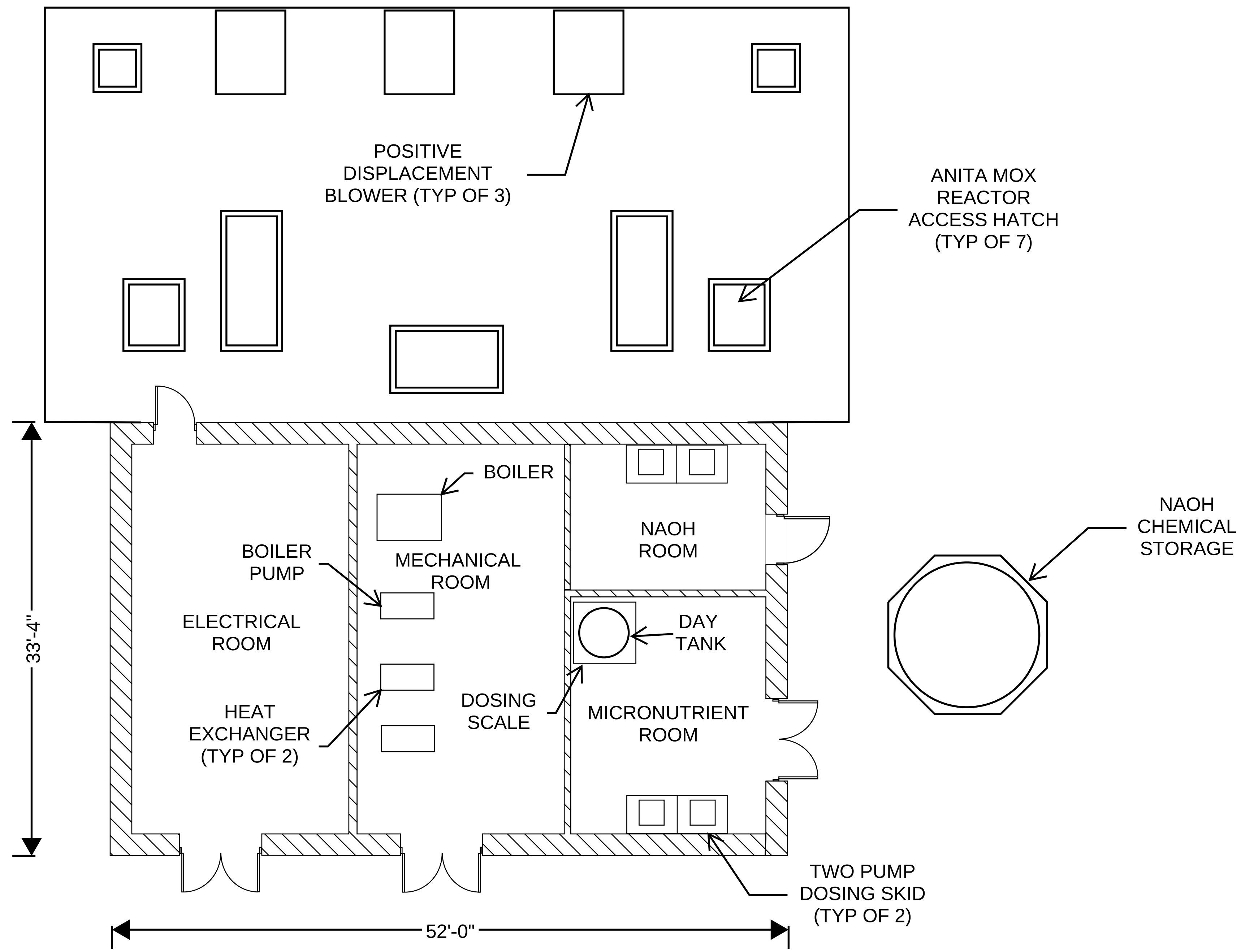
The reactors, shown in Figure 2-5, consist of aeration piping and a top entering vertical mixer in addition to the Anita™ Mox growth media and two media retention sieves. The top slab of the Centrate Equalization Basin and Sidestream Deammonification Reactors 1 and 2 consists of various access hatches and penetrations for ventilation and odor control ductwork. Additionally, three positive displacement blowers and the two above-mentioned top entering vertical mixers will be located on the slab directly above the sidestream reactors. Table 2-10 and Table 2-11 summarize the mixer and blower design criteria for the Anita™ Mox process.

Table 2-10 Sidestream Deammonification Mixer Design Criteria

PARAMETER	DESIGN CRITERIA
Mixer Type	Vertical
Quantity	2 (1 per basin)
Motor Rating, Hp	15
Maximum Motor Speed, rpm	1800
Minimum Impeller Diameter, in	106 (Stage one) 130 (Stage two)
Mixer Speed	Variable

Table 2-11 Sidestream Deammonification Blower Design Criteria

PARAMETER	DESIGN CRITERIA
Blower Type	Positive Displacement
Quantity	3
Discharge Pressure at Outlet Flange, psig	10
Minimum Capacity at Rated Discharge Pressure, scfm	520
Drive Motor Rating, Hp	40
Maximum Motor Speed, rpm	3600
Blower Speed	Variable



3/32" = 1'-0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 3 - SECONDARY AND SIDESTREAM TREATMENT
SIDESTREAM DEAMMONIFICATION BUILDING LAYOUT
FIGURE 2 - 4

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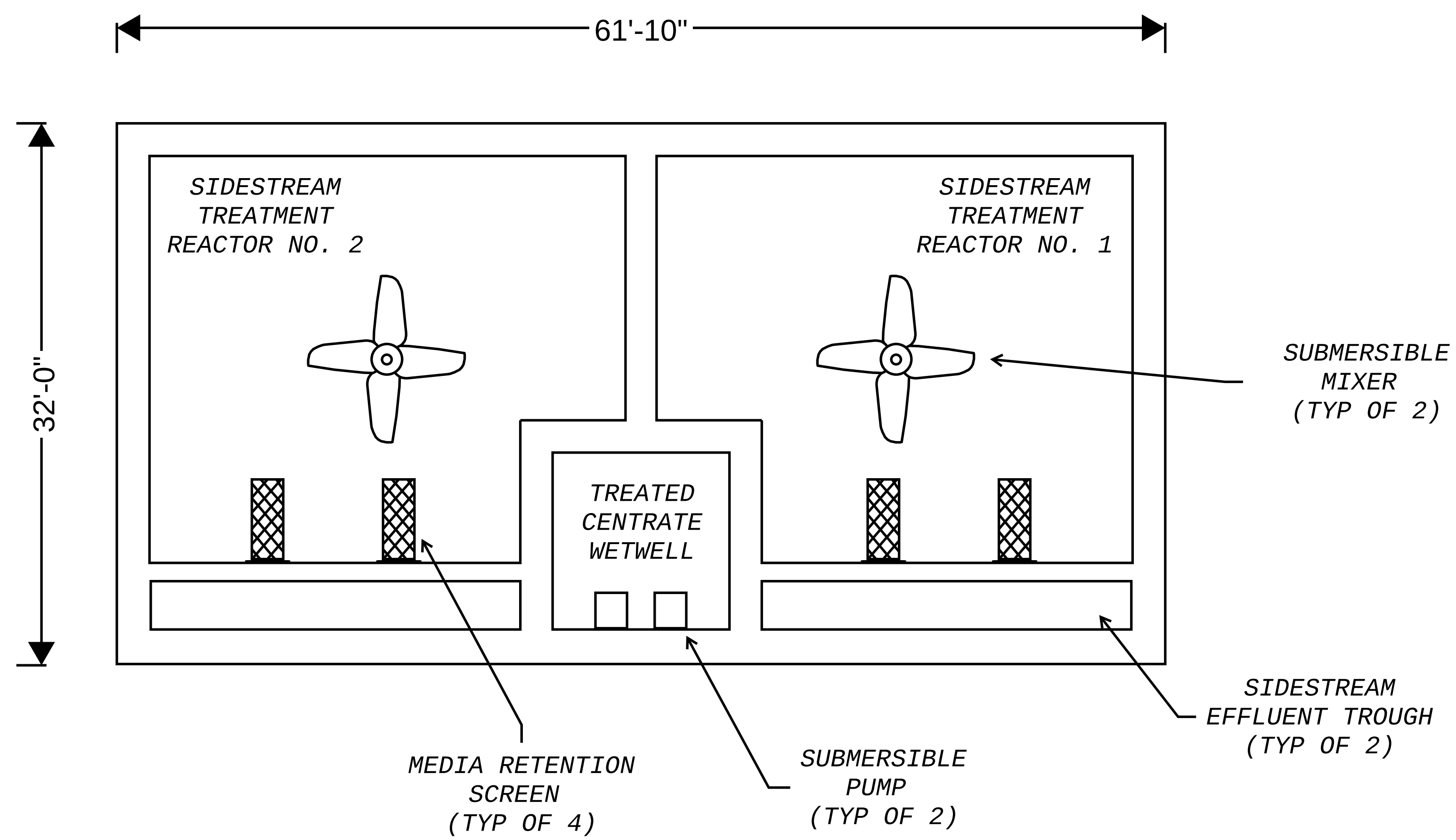


JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 3 - SECONDARY AND SIDESTREAM TREATMENT
SIDESTREAM DEAMMONIFICATION REACTORS
FIGURE 2 - 5

BLACK & VEATCH

3.0 Cost Analysis

Preliminary capital and O&M costs were developed for the secondary treatment and sidestream deammonification processes described in Section 2. The estimates are in 2020 dollars.

3.1 SUMMARY OF CAPITAL COSTS

The basis of design presented in this TM was used to develop a preliminary opinion of probable construction cost for secondary and sidestream deammonification facilities. The costs presented below do not include cost of electrical, sitework, I&C, ELA, or contingencies. These costs will appear as line items in the overall opinion of probable construction cost presented in the Facility Plan Report.

3.1.1 Secondary Treatment

Estimated capital cost for all structures and equipment associated with secondary treatment is summarized in Table 3-1.

Table 3-1 Secondary Treatment Capital Cost

	CAPITAL COST ($)	
	FC ALT 1 – Retrofit Existing Clarifiers	FC ALT 2 – Construct New Clarifiers
BNR Basin	$20,254,000	$20,254,000
Final Clarifiers	$7,757,000	$6,031,000
Final Sludge Pump Station	$5,534,000	$6,304,000
Basin Blower Building	$4,165,000	$4,165,000
Total	**$37,710,000**	**$36,754,000**

- Capital costs presented in January 2020 dollars.
- Costs exclude electrical, site, I&C, ELA and contingencies.
- Presented capital costs are conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

The geometry of the MCR WWTP BNR Basin was modeled after the BNR Basin at the THC WWTP, but the required volume was calculated independently. Concrete costs were calculated by scaling the basin by its new volume, equipment costs were added separate. The capital cost of the ferric chloride system utilized in the event of a BNR upset is also included in the BNR Basin capital cost. The Basin Blower Building capital cost was developed by increasing the building footprint of the THC WWTP Basin Blower Building to account for slightly larger blowers and a separate chemical feed room.

Final clarifier costs were determined by scaling the THC WWTP clarifier costs by the new quantity and diameters required at MCR WWTP. To retrofit the clarifiers in FC Alt 1, only the equipment cost for the THC WWTP final clarifiers were added to the total. The new Final Sludge Pump Station associated with FC Alt 1 was modeled after the THC WWTP with reduced equipment costs, while updates to the existing pump station were determined by applying a scaled unit price determined for the THC WWTP RAS and WAS pumps. The FC Alt 2 Final Sludge Pump Station was modeled after the THC WWTP Final Sludge Pump Station with a slightly larger footprint to accommodate larger pumps.

3.1.2 Sidestream Deammonification

Estimated capital cost for the Sidestream Deammonification Building and associated equipment is summarized in Table 3-2. Note that capital cost of the associated Centrate Equalization Basin is included in TM 6 – Biosolids Treatment.

Table 3-2 Sidestream Deammonification Capital Cost

	CAPITAL COST ($)
Sidestream Deammonification Building	$4,911,000

- Capital costs presented in 2020 dollars.
- Costs exclude electrical, site, I&C, ELA and contingencies.
- Presented capital costs are conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

The capital cost associated with the Sidestream Deammonification Building at the MCR WWTP was derived almost directly from the Sidestream Treatment Building at the THC WWTP. For this study, the capital cost associated with the Centrate Equalization Basin was stripped from the THC Sidestream Treatment Building and moved to the Dewatering Building cost. Additionally, the building footprint was enlarged to the accommodate separate rooms for each chemical associated with the AnitaMox process.

3.2 SUMMARY OF OPERATION AND MAINTENANCE COSTS

Operations and maintenance costs include the cost of power, chemicals, operating labor, and general maintenance. O&M costs are calculated based on annual average conditions and solids production. The estimates are in 2020 dollars.

3.2.1 Secondary Treatment

O&M costs developed for secondary treatment are presented in Table 3-3.

Table 3-3 Secondary Treatment O&M Annual Cost Estimates

	BNR BASIN	BASIN BLOWER BUILDING	FINAL CLARIFIERS		FINAL SLUDGE PUMP STATION	
			FC ALT 1	FC ALT 2	FC ALT 1	FC ALT 2
Power	$77,000	$365,000	$1,000	$1,000	$37,000	$35,000
Labor	$44,000	$26,000	$18,000	$18,000	$35,000	$35,000
Equipment Maintenance	$13,000	$32,000	$26,000	$23,000	$10,000	$10,000
Chemicals	$80,000	-	-	-	-	-
Total	**$214,000**	**$423,000**	**$45,000**	**$42,000**	**$82,000**	**$80,000**

3.2.2 Sidestream Deammonification

O&M costs developed for the Sidestream Deammonification Building are presented in Table 3-4. Note that O&M cost of the associated Centrate Equalization Basin is included in TM 6 – Biosolids Treatment.

Table 3-4 Sidestream Deammonification O&M Annual Cost Estimates

	SIDESTREAM DEAMMONIFICATION BUILDING
Power	$19,000
Labor	$26,000
Maintenance	$33,000
Chemicals	$157,000
Total	**$235,000**

4.0 Summary of Findings and Recommendations

4.1 SECONDARY TREATMENT

For secondary treatment, it is recommended that a four-train plug flow S2EBPR BNR Basin be constructed. A nearby Basin Blower Building will house the five high speed gearless turbo blowers required to meet the aeration demands of the BNR Basin.

The recommendation for configuration of final clarifiers and associated pump station is contingent upon the site optimization carried out in TM 8 – Site Optimization & MOPO. The following sections outline the recommendations for each alternative pending results of TM 8 – Site Optimization & MOPO.

4.1.1 FC Alternative 1 – Retrofit Existing Final Clarifiers

Three (3) new 130-foot final clarifiers will be constructed in addition to replacing the clarifier equipment of the 2 existing final clarifiers. For site optimization purposes, the existing Final Sludge Pump Station will be left in service, but not expanded for the newly constructed final clarifiers. Rather, a new pump station will be constructed to service the new clarifiers.

4.1.2 FC Alternative 2 – All New Final Clarifiers

For site optimization purposes, the two existing final clarifiers and associated Final Sludge Pump Station will be taken out of service and demolished. Four (4) new 145-foot final clarifiers and 1 common Final Sludge Pump Station will be constructed in the location recommended in TM 8 – Site Optimization & MOPO.

4.2 SIDESTREAM DEAMMONIFICATION

For sidestream deammonification, a Veolia Anita™ Mox system is recommended for installation. The two sidestream deammonification reactors should be located below grade to minimize heat loss. Unlike THC WWTP, the above grade Sidestream Deammonification Building that houses all ancillary equipment and chemicals necessary to support the Anita™ Mox process should be located directly adjacent to the below grade reactors. The above grade structure at THC WWTP is located directly on top of the deammonification reactors to reduce footprint, which is not a concern at MCR WWTP.

DRAFT

MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 4
Auxiliary Wet Weather Treatment

JCW NO. MCR1-BV-17-12
BV PROJECT 403165

PREPARED FOR

JOHNSON COUNTY KANSAS
Wastewater

SEPTEMBER 17, 2020

BLACK & VEATCH

Table of Contents

LIST OF TABLES

LIST OF FIGURES

Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CANDO	Coupled Aerobic-anoxic Nitrous Decomposition Operation
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEA	Cost Effective Analyses
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
CIPP	Cured-in-place Pipe
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSBR	Continuous Sequencing Batch Reactor
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
D	
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry-weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered

Abbreviation	Meaning
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand
ffCOD	Flocculated Filtered Chemical Oxygen Demand
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
ft	Feet
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
H	
HB	Hallbrook Facility
HDD	Horizontal Directional Drilling
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration
IC	Internal Combustion
IFAS	Integrated Fixed-Film Activated Sludge
in	Inches
IND	Industrial
INF	Influent
IP	Intellectual Property
IPS	Influent Pump Station
IR	Irrigation Use
IRR	Irrigation
IW	Industrial Water Supply Use
J	
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDHE	Kansas Department of Health and Environment
K_e	Light Extinction Coefficient
kWh	Kilowatt-hour
L	
L	Length, Liter
lb	Pound
LF	Linear Feet
LOMR	Letter of Map Revision
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter
mgd	Million Gallons per Day
min	Minute, Minimum

Abbreviation	Meaning
mJ	Millijoules
MLE	Modified Ludzack Ettinger
MLSS	Mixed Liquor Suspended Solids
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow
mmBtu	Million British Thermal Units
MOPO	Maintenance of Plant Operations
mpg	Miles per Gallon
MPN	Most Probable Number
µg/L	Micrograms per Liter
N	
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NFIP	National Flood Insurance Program
NH3-N	Total Ammonia
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
NPV	Net Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow
PE	Primary Effluent
PFE	Primary Filtered Effluent

Abbreviation	Meaning
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow
PIF	Peak Instantaneous Flow
PLC	Programmable Logic Controller
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day
pph	Pounds per Hour
PPI	Producer Price Index
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet-weather Flow
Q	
Q	Flow
R	
RAS	Return Activated Sludge
RAS	
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
Sec	Second, Secondary
SF	Square Foot
SG	Specific Gravity
SLR	Solids Loading Rate

Abbreviation	Meaning
SMP	Stormwater Management Program, Shawnee Mission Park Pump Station
SND	Simultaneous Nitrification/ Denitrification
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station
SRT	Sludge Retention Time
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
TYP	Typical

Abbreviation	Meaning
U	
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VFA	Volatile Fatty Acids
VFAs	
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	Width
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation –Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WLWater LevelWK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

The purpose of this technical memorandum (TM) is to summarize the conceptual design of the auxiliary wet weather treatment facilities at the Mill Creek Regional (MCR) Wastewater Treatment Plant (WWTP). For the purposes of the MCR WWTP, auxiliary wet weather treatment is defined as flows between the peak secondary flow and peak day flow that are treated separately from the peak secondary flow. This TM includes a discussion of available treatment technologies, design criteria of the selected technology, footprint and layouts of the selected technology, capital costs, and operational and maintenance (O&M) costs.

This TM is one in a series of technical memoranda that will be incorporated into a Facility Plan report summarizing a future expansion of the MCR plant. Additional treatment processes and site optimization of these treatment facilities will be outlined in future TMs.

1.1 BACKGROUND

Prior to this Facility Plan for MCR, an extensive alternative analysis was done for the Tomahawk Creek (THC) WWTP Expansion. The results of this analysis can be used to inform the planning of the MCR Expansion. The THC WWTP is a good comparison because it is a similarly-sized facility (19 million gallons per day (mgd) annual average (AA) flow), with similar wastewater characteristics, is owned and operated by JCW and has actual market costs for treatment technologies provided by a Contractor.

In August of 2014, Johnson County Wastewater (JCW) retained Black & Veatch (BV) for the project definition phase of the THC WWTP Expansion. The primary objective of the project definition phase was to confirm, through alternative development and evaluation, the optimal and proven treatment strategies throughout the WWTP for nutrient removal to meet current and anticipated future NPDES limits for design flows. Evaluation of these alternatives consisted of utilizing the JCW's triple bottom line (TBL) approach to evaluate non-economic factors in addition to developing capital and operating costs for each alternative. Each treatment process evaluation was presented to JCW to select a recommended technology to be carried forward through design and construction.

After the project definition phase, the THC WWTP Expansion was continued into detailed design, followed by construction. The construction is scheduled to be completed in 2021. During the detailed design phase, some of the selected treatment technologies were reevaluated and eventually revised as part of a value engineering effort. The treatment technologies that were part of the final design and eventually carried into construction serve as a valuable comparison for the MCR WWTP.

From TM 1, the design flows for the MCR WWTP were established and are shown in Table 1-1. It should be noted that the auxiliary treatment processes will be sized to handle a total of 63 mgd. This is three times the design AA flow (3Q). During peak day conditions, the peak secondary flow will be treated through the plant, and the 63 mgd associated with wet weather flow will be treated via auxiliary treatment.

Table 1-1 MCR Design Flows

	DIURNAL LOW AA STARTUP	AA STARTUP	AA ULTIMATE	MAX MONTH	PEAK SECONDARY	PEAK DAY
MCR Design Flows (mgd)	6.0[1]	12.0[2]	21.0	31.5	63.0	126.0

[1] Historically this is 1/2 of the diurnal high (AA startup).
[2] Flow projection based on TM 1, Figure 2-3, and Year 2034 startup, assuming 1% growth.

1.1.1 Existing Wet Weather Treatment at MCR WWTP

Currently at MCR, the wet-weather pumps in the Influent Pump Station (IPS) convey the excess flow to the head of the partially-mixed Lagoon Aeration Cells (Cells 3 and 4) during wet weather events beyond 24 mgd (equivalent to two times the design AA flow). From the head of Lagoon Cells 3 and 4, flow makes its way through Cells 5 and 6 before ending up in Cell 8. Cell 8 effluent enters the Plant Effluent Junction Box, where it is recombined with treated effluent from the mechanical plant UV Disinfection facility. From the Plant Effluent Junction Box, flow is sent by gravity through the effluent tunnel to the Kansas River.

1.2 AUXILIARY TREATMENT

Rather than starting a completely new and independent evaluation of available technologies for flows exceeding the Peak Secondary Flow at MCR WWTP, it was decided to build off the evaluation completed for the THC WWTP Expansion that is currently under construction. In selecting auxiliary treatment alternatives for the evaluation at THC WWTP, a matrix was developed to summarize and screen five technologies against initial criteria. After this initial evaluation, two alternatives — microsand ballasted flocculation (ACTIFLO®) and compressible media filtration (CMF) — were carried forward for a more detailed evaluation. The primary driver for these technologies was a small footprint design. The THC WWTP site is limited in space when compared to MCR. During the value engineering effort that was part of the THC detailed design, the auxiliary treatment technology was switched to cloth media disk filtration (disk filters). There were two primary drivers for this change: lower capital cost and additional performance data.

The auxiliary treatment technology change at THC WWTP saved an estimated $14,000,000 for the construction cost of the project. The majority of the savings was due to equipment costs and constructability benefits.

In addition to the economic savings, disk filters were not carried forward to the detailed evaluation during the initial technology evaluation due to the lack of operating full-scale facilities for direct filtration of wet weather primary influent flows. In the months between the preliminary project definition phase and the value engineering effort, full-scale facilities were placed in operation and the performance data collected from those facilities showed good results; therefore, cloth disk filters became a viable and proven alternative for wet weather primary influent and became the selected auxiliary treatment technology for THC WWTP.

For the purposes for MCR, cloth disk filters will be the basis of design. ACTIFLO® and CMF will not be further evaluated in this TM.

2.0 Basis of Design Criteria

One key difference between the THC WWTP and the MCR WWTP is the future permit limits at each site. The THC limit for Total Phosphorus (TP) is 0.5 milligrams per liter (mg/L). The MCR limit for TP is 1.0 mg/L. To achieve the TP permit limit at THC, tertiary treatment is required; as a result, the same filter complex is designed to provide both tertiary treatment and auxiliary treatment of wet weather flows. The filter mode of operation is based on flows to the plant. When flows are up to three times the annual average flow (3Q), the filters operate in tertiary treatment mode. When flows are between 3Q and 6Q, the filters operate in a "dual-purpose" mode, treating 3Q tertiary flow and 3Q auxiliary flow at the same time. Once flow exceeds 6Q, the filters operate strictly in an auxiliary treatment mode to provide peak flow treatment.

At MCR, tertiary treatment is not expected to be required to meet permit since the effluent TP limit is less stringent; however, since the cloth disk filters will already be installed for wet weather treatment, there is a "dual-purpose" benefit of using the installed filters for tertiary treatment during dry weather flows. There will be minimal capital cost impacts associated with using the installed filters.

If, in the future, the TP permit limits at MCR are changed to be more restrictive and tertiary treatment is required during dry weather flows, more filter capacity can be added as needed by constructing a new filter complex, or simply adding on to the existing wet weather filter complex. The auxiliary treatment filter complex will be located on site such that there is room for future filter cells. The physical location of treatment processes on site will be discussed in TM 8.

A schematic of the dual-purpose filter design is presented in Figure 2-1. This schematic represents when the system is operating in Tertiary Mode and when the system is operating in Auxiliary Mode. Treatment processes that are depicted but not covered in this TM include primary, secondary, and disinfection treatment. These treatment processes are covered in other TMs and are in this schematic for representation purposes only.

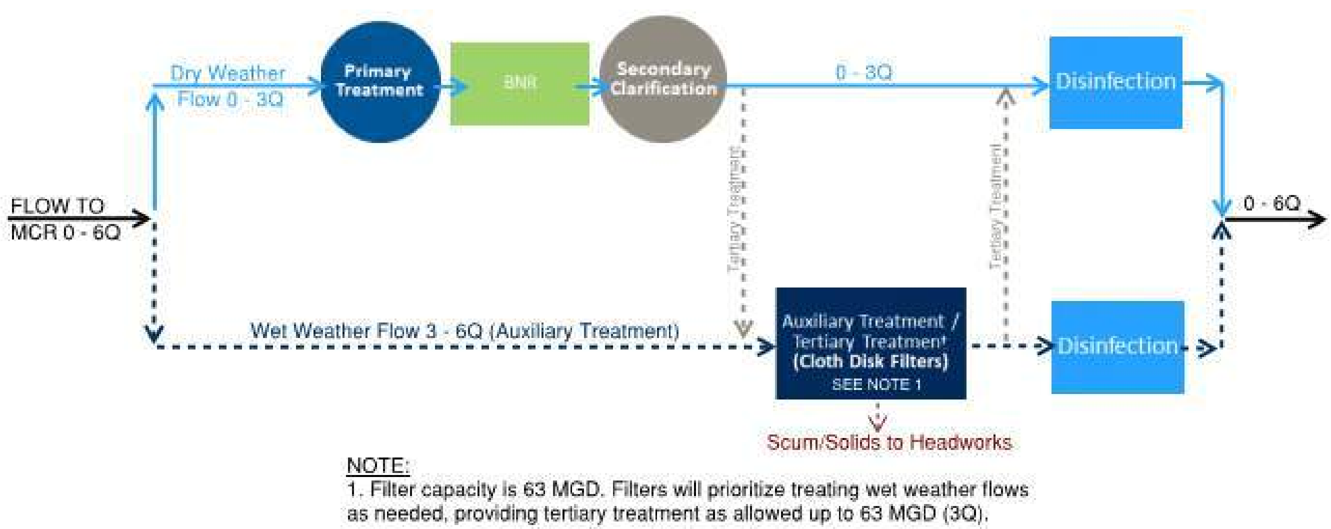


Figure 2-1 Dual-Purpose Filter Schematic

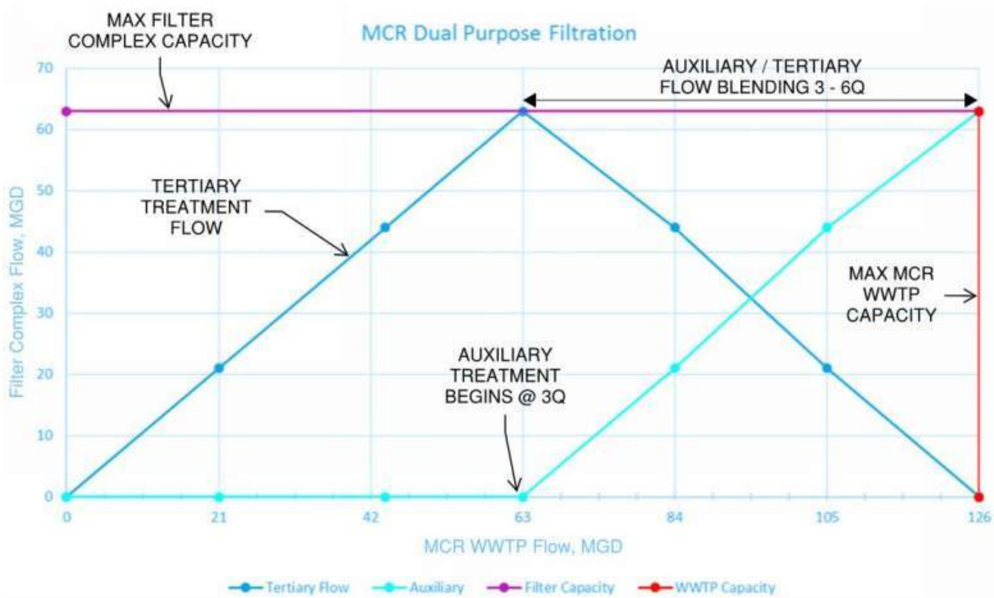


Figure 2-2 Dual-Purpose Filter Operation

Figure 2-2 indicates that the filter system will be operating in tertiary treatment mode as the dry weather flow increases up to three times the design average flow 3Q. As wet weather flow increases the total flow above 3Q, the filters will operate in auxiliary treatment mode as required to provide filtration to all flow that does not go through secondary treatment. As indicated in the figure, the maximum capacity of the filter complex is 63 mgd.

2.1 DUAL PURPOSE DESIGN CRITERIA

There are a few pile cloth disk filters on the market; however, Aqua-Aerobics is the only manufacturer that has a filter unit appropriate for a facility the size of MCR. In addition, Black & Veatch's experience with piloting and the design of pile cloth disk filters is only with Aqua-Aerobics. For these reasons, the Aqua-Aerobics Cloth MegaDisk Filters are recommended at MCR.

The MegaDisk Cloth Filters are a proprietary process offered by Aqua-Aerobic Systems, Inc. as shown in Figure 2-3. This model unit is the largest diameter disk offered by Aqua-Aerobics, and has a small footprint with a high total suspended solids (TSS) removal rate.

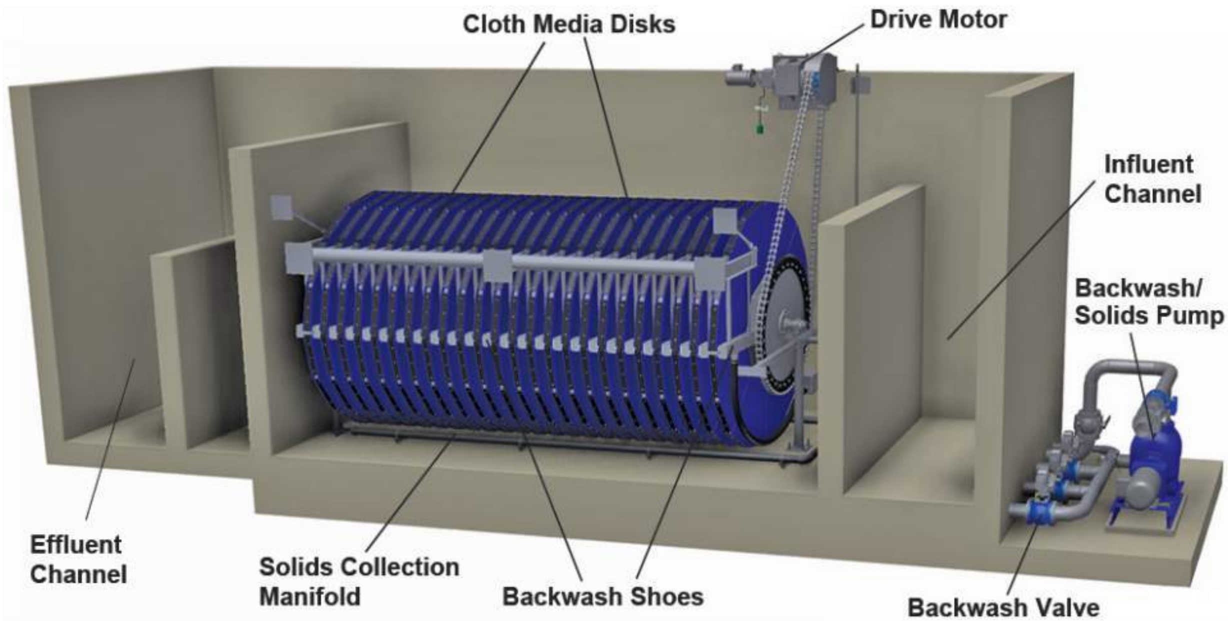


Figure 2-3 Aqua-Aerobic Systems, Inc. MegaDisk Cloth Media Filter

The filter complex will have a common influent distribution channel where flow is directed to each on-line filter through means of a filter influent isolation gate. Influent wastewater flows from the influent channel into each filter cell over the influent weir, completely submerging the static cloth media disks under all operating conditions. As influent passes through the cloth on both sides of the disk, solids accumulate on the cloth media and a solids mat is formed. Filtrate is collected in the center drum, and is then directed to the effluent chamber and over the effluent weir. The design criteria for the dual-purpose disk filters is outlined in Table 2-1. The design criteria are based upon the Aqua-Aerobics MegaDisk filter.

Table 2-1 Cloth Disk Filters Design Criteria

PARAMETER	DESIGN CRITERIA
Total Number of Filter Cells	6
Filter Cell Submerged Filtration Area, SF	2,582
Number of Disks per Unit	24
Diameter per Disk, ft	10
Auxiliary Mode	
Peak Flow Rate, mgd	63
Maximum Number of Filters in Operation	5
Average Influent Total Suspended Solids, mg/L	52
Maximum Influent Total Suspended Solids, mg/L	61
Maximum Operating Hydraulic Loading Rate[1] (HLR), gpm/SF	3.39

PARAMETER	DESIGN CRITERIA
Maximum Solids Loading Rate[1] (SLR), ppd/SF	2.47
Tertiary Mode	
Average Flow Rate, mgd	21
Peak Flow Rate, mgd	63
Maximum Number of Filters in Operation	5
Average Influent Total Suspended Solids, mg/L	10
Maximum Influent Total Suspended Solids, mg/L	30
Maximum Operating Hydraulic Loading Rate[1], gpm/SF	3.39
Maximum Solids Loading Rate[1], ppd/SF	1.22
Filter Drive Motor, hp	5
[1]Max loading rates are with 1 unit out of service.	

The disk filters are designed to achieve the max HLR and SLR at peak wet weather conditions with one cell out of service. Aqua-Aerobics recommends a maximum HLR of 4 gpm/SF and a maximum SLR of 12 ppd/SF. The SLR is well below the manufacturer recommended rate, meaning the disk filters are hydraulically limited due to the dilute nature of wet weather flow.

As the filter operates, eventually it begins to accumulate solids in the filter media, i.e. blinding, and the level within each filter bay rises to a preset point where a PLC automatically initiates the backwash cycle. During the backwash cycle, the filter remains in service. One-third of the disks are backwashed at a time by rotating the entire filter assembly. During rotation, solids are vacuumed from the surface by backwash shoes that pull filtered water from inside the filter disk. The backwash shoes make firm contact with the cloth media, maximizing effective cleaning while filtration continues on the other two-thirds of the disks without interruption. The backwash is pumped with the suction style backwash/solids wasting pumps that send flow to the plant drain system which eventually takes it to the head of the plant. Due to the vertical orientation of the cloth media, heavier solids settle to the bottom of the tank. These solids are pumped on an intermittent basis by opening a valve and using each filter cells' backwash/solids wasting pump, which is provided by the manufacturer. Scum in each of the filter cells flows over a weir, where it eventually collects in a common scum channel and flows by gravity to the plant drain system, which eventually brings it back to the head of the plant. The backwash/solids wasting pumps design criteria are outlined in Table 2-2.

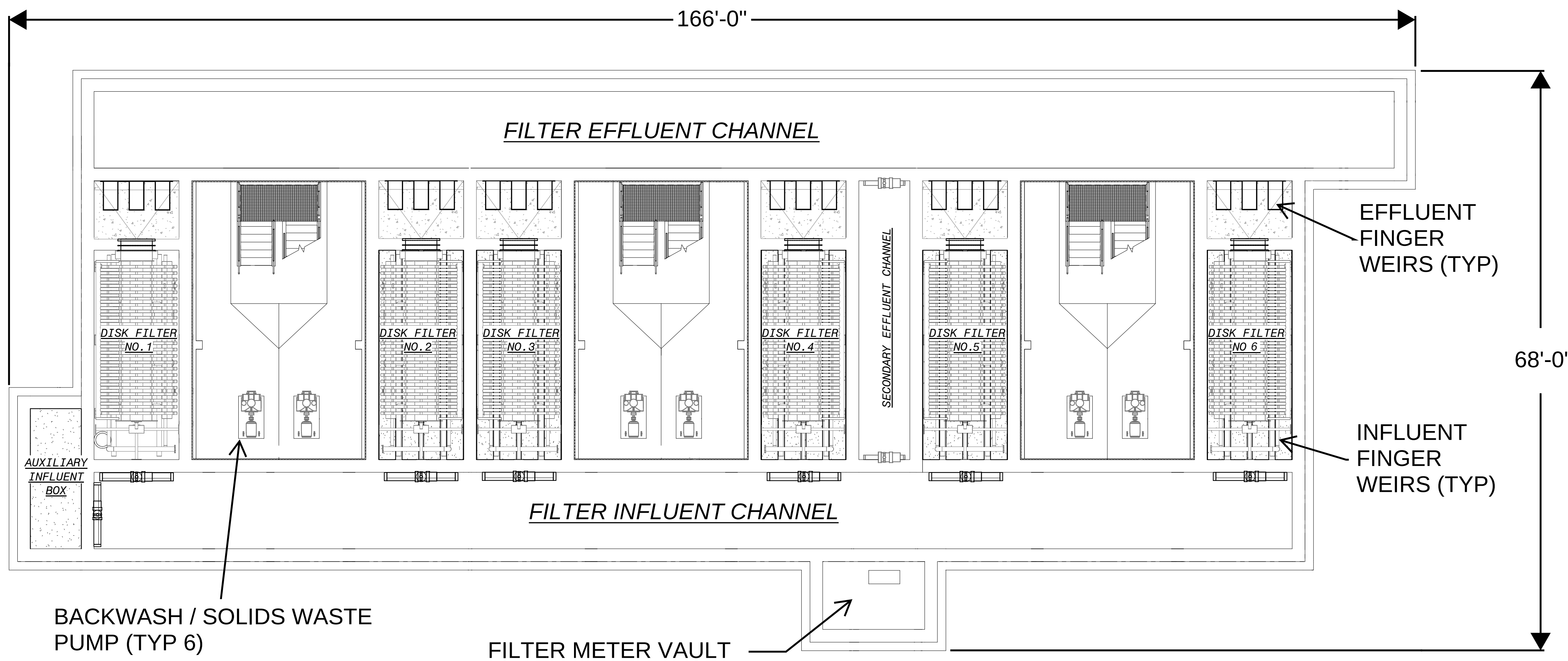
Table 2-2 Backwash/Solids Wasting Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Pumps	6 (1 pump per filter cell)
Type	Self-priming centrifugal
Pumping Capacity, gpm	780
Pump Motor, Hp	20

All filters are provided with an installed filter drum and backwash motor. Additional uninstalled spares can be provided for the event of an equipment failure. To bring additional filter cells on-line to match influent flow conditions, the influent isolation gates will be opened by the PLC control system based on a flow set point and the filters will commence filtering flows automatically. This control can also be tied to the filters' internal high-level alarms.

Figure 2-4, Figure 2-5, and Figure 2-6 show plans and a section view of the proposed dual-purpose filter complex at MCR.

Figure 2-4 shows a secondary effluent channel between Disk Filters 4 and 5. This secondary effluent channel is used only when secondary effluent is diverted from going through the filters to free up capacity for wet weather auxiliary treatment. Secondary effluent enters the filter meter vault and flows into the secondary effluent channel below the filter influent channel. During dry weather flows, the secondary effluent then flows over the filter influent isolation gate into the filter influent channel for distribution to the filter cells. The water level in the secondary effluent channel is controlled by a modular level control gate; during wet weather flow events, this gate adjusts its level to divert the appropriate amount of secondary effluent directly to disinfection via the filter effluent channel. This automatic modular control allows all auxiliary wet weather flow through the filters.

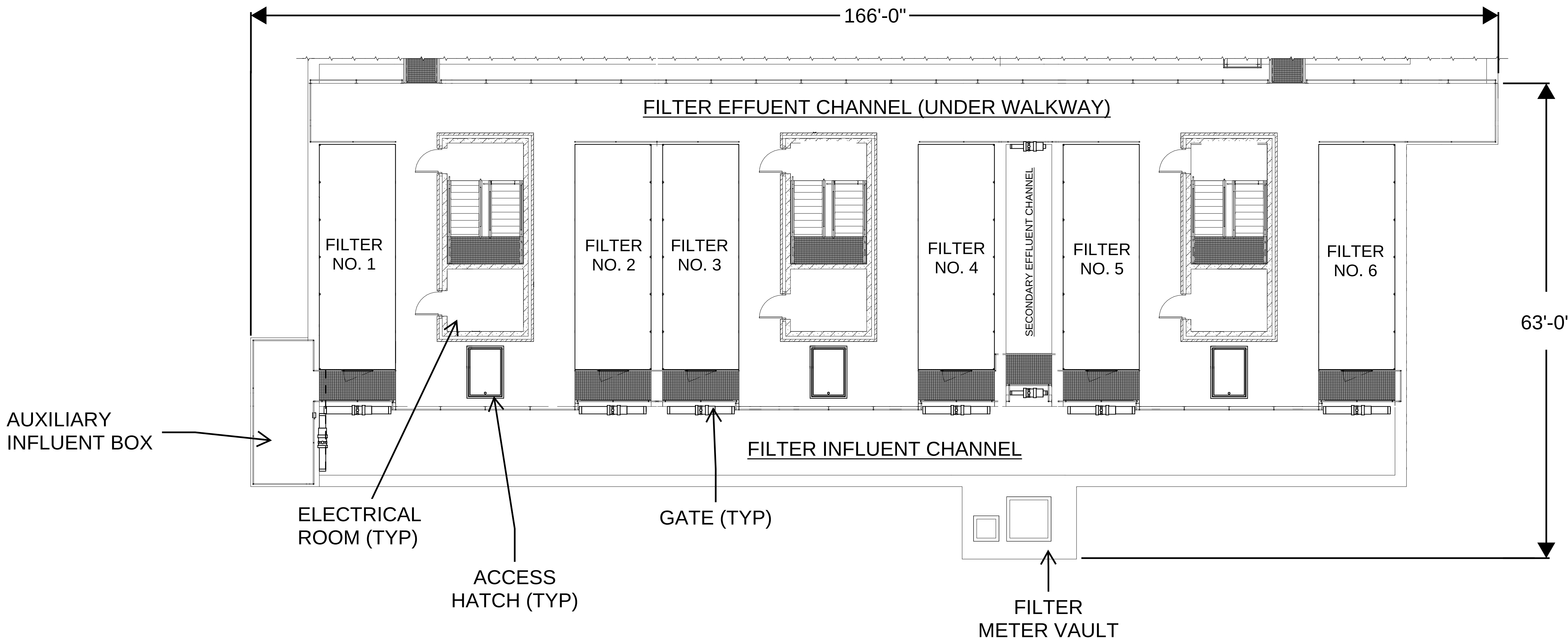


SCALE 1/8" = 1' 0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 4 - AUXILIARY WET WEATHER TREATMENT
FILTER COMPLEX LOWER LEVEL
FIGURE 2 - 4

BLACK & VEATCH

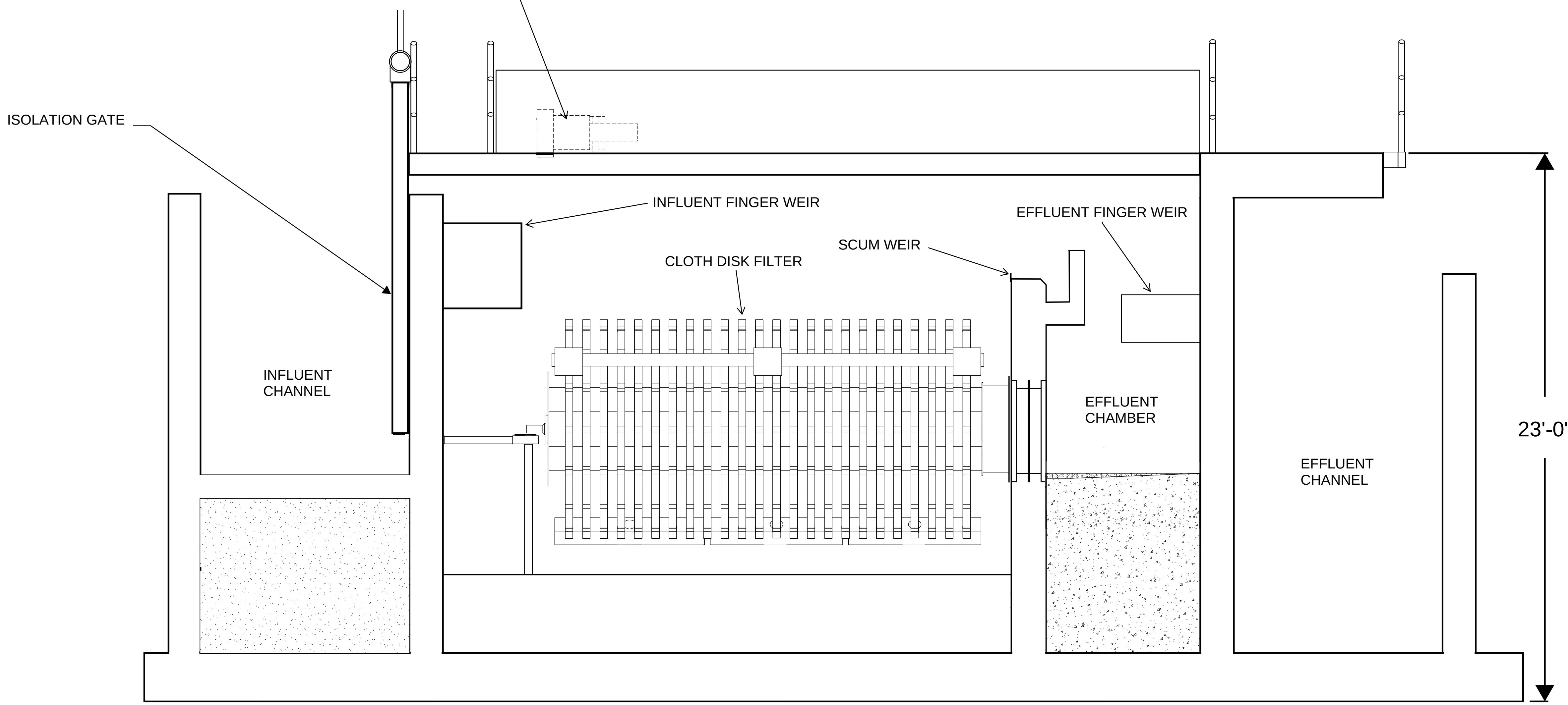


SCALE 1/8" = 1' 0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 4 - AUXILIARY WET WEATHER TREATMENT
FILTER COMPLEX UPPER LEVEL
FIGURE 2 - 5

BLACK & VEATCH



SCALE 3/8" = 1' 0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 4 - AUXILIARY WET WEATHER TREATMENT
FILTER COMPLEX SECTION
FIGURE 2 - 6

BLACK & VEATCH

3.0 Cost Analysis

Preliminary capital and O&M costs were developed for the auxiliary wet weather treatment system as described in Section 2. The estimates are in 2020 dollars.

3.1 SUMMARY OF CAPITAL COSTS

All auxiliary wet weather treatment equipment will be located at the filter complex. An opinion of probable construction cost of this filter complex is shown in Table 3-1. The costs presented below do not include the cost of electrical, sitework, instrumentation and control, engineering, legal, administration (ELA), or contingencies. These costs will appear as line items in the overall opinion of probable construction cost presented in the Facility Plan Report.

Table 3-1 Auxiliary Treatment Opinion of Probable Capital Construction Cost

	CAPITAL COSTS ($)
Filter Equipment Installed Cost	$6,300,000
Filter Complex Structure Cost	$2,951,000
Filter Complex Total Cost	**$9,251,000**

- Capital costs are presented in January 2020 dollars.
- Costs exclude electrical, site, I&C, ELA, and contingency costs
- OPCCs are conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

The Filter Complex at the MCR WWTP was modeled after the Filter Complex at the THC WWTP. The THC WWTP Filter Complex was designed to treat more flow than the MCR WWTP Filter Complex. As such, the THC WWTP Filter Complex has two more filters than what is required at the MCR WWTP. Having two less filters reduces both the equipment cost and the structure cost since the overall structure footprint is smaller when two filters are removed. Both the equipment and structure costs at the THC WWTP were scaled to account for the reduced quantity of filters. In addition, the THC WWTP costs are based on 2018 dollars. To represent the most accurate cost at the MCR WWTP, the costs have been inflated to 2020 dollars using the Engineering News Record (ENR) construction cost index.

3.2 SUMMARY OF OPERATIONAL AND MAINTENANCE COSTS

Operations and maintenance costs include the cost of power, chemicals, operating labor, and general maintenance. O&M costs are based on annual average conditions and solids production. The power demand is based on the presented design criteria and manufacturer data. The labor costs are based on a Black & Veatch estimate of hours per week of total labor associated with the Filter Complex. The equipment maintenance cost is 2 percent of the equipment capital cost. The O&M cost summary is presented in Table 3-2.

Table 3-2 Auxiliary Treatment O&M Annual Cost Estimate

	COSTS ($)
Power	31,000
Labor	18,000
Equipment Maintenance	126,000
Chemicals	11,000
Total	**$186,000**

Looking at Table 3-2, the biggest maintenance cost is the filter media replacement. Aqua-Aerobics estimates a seven-year life for media under tertiary loading. Costs associated with media replacement for six filters is similar to the annual equipment maintenance cost over seven years. Table 3-2 also indicates an annual chemical cost of approximately $11,000. This cost is due to a small dose of sodium hypochlorite that is periodically applied to filters. The hypochlorite is applied as a preventative maintenance measure to minimize buildup on the cloth media filters. Maintaining relatively clean filter media helps maintain pump efficiency. This cleaning system is designed for periodic operation; however, each filter can also be cleaned by continuous exposure at lower doses. During the periodic exposure, the filter is offline for up to three hours.

4.0 Summary of Findings and Recommendations

4.1 AUXILIARY WET WEATHER TREATMENT

It is recommended that six cloth disk filter units be installed at MCR. The recommended filter complex is essentially three pairs of two filter cells that share a filter pump station. Each of the three pump stations houses a pair of filters and backwash/solids wasting pumps. This configuration is similar to the Filter Complex at THC WWTP. The filter technology is recommended to be the Aqua-Aerobic MegaDisk Cloth Disk Filter. These filters are primarily being installed for auxiliary wet weather treatment purposes, which include flow events above three and up to six times the design average flow. The recommended filter capacity is 63 mgd.

The filters are only required for use during wet weather events; however, there is an opportunity to also use the filters for tertiary treatment use during dry weather flows, which would create a dual-purpose filter facility. Although tertiary treatment is not expected to be needed to meet the future permit limits at MCR, it still provides a benefit to the overall level of treatment. Since the filters will already be installed for auxiliary purposes, it is recommended that this dual-purpose benefit is utilized, especially since it has minimal capital impact.

If the future TP permit limit at MCR is changed at some point to be more restrictive and tertiary treatment is needed, additional filter cells could be easily added. If additional filter cells are needed, they can be added to the filter complex discussed in this TM, or a separate additional filter complex can be added. Although the physical location on the MCR site (where the filter complex will be located) will be determined in TM 8 - Site Optimization and MOPOs, adequate room around the filter complex will be provided, allowing for future expansion.

DRAFT

MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 5
Disinfection Treatment

JCW NO. MCR1-BV-17-12
BV PROJECT 403165

PREPARED FOR

JOHNSON COUNTY
KANSAS
Wastewater

SEPTEMBER 17, 2020

BLACK & VEATCH

Table of Contents

LIST OF TABLES

LIST OF FIGURES

Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CANDO	Coupled Aerobic-anoxic Nitrous Decomposition Operation
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEA	Cost Effective Analyses
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
CIPP	Cured-in-place Pipe
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSBR	Continuous Sequencing Batch Reactor
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
D	
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry-weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered

Abbreviation	Meaning
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand
ffCOD	Flocculated Filtered Chemical Oxygen Demand
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
ft	Feet
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
gph	Gallons per Hour
gpm	Gallons per Minute
gpy	Gallons per Year
H	
HB	Hallbrook Facility
HDD	Horizontal Directional Drilling
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
Hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration
IC	Internal Combustion

Abbreviation	Meaning
IFAS	Integrated Fixed-Film Activated Sludge
in	Inches
IND	Industrial
INF	Influent
IP	Intellectual Property
IPS	Influent Pump Station
IR	Irrigation Use
IRR	Irrigation
IW	Industrial Water Supply Use
J	
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDHE	Kansas Department of Health and Environment
K_e	Light Extinction Coefficient
kWh	Kilowatt-hour
L	
L	Length, Liter
lb	Pound
LF	Linear Feet
LOMR	Letter of Map Revision
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter

Abbreviation	Meaning
mgd	Million Gallons per Day
min	Minute, Minimum
mJ	Millijoules
MLE	Modified Ludzack Ettinger
MLSS	Mixed Liquor Suspended Solids
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow
mmBtu	Million British Thermal Units
MOPO	Maintenance of Plant Operations
mpg	Miles per Gallon
MPN	Most Probable Number
µg/L	Micrograms per Liter
N	
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NFIP	National Flood Insurance Program
NH3-N	Total Ammonia
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
PV	Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow

Abbreviation	Meaning
PE	Primary Effluent
PFE	Primary Filtered Effluent
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow
PIF	Peak Instantaneous Flow
PLC	Programmable Logic Controller
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day
pph	Pounds per Hour
PPI	Producer Price Index
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet-weather Flow
Q	
Q	Flow
R	
RAS	Return Activated Sludge
RAS	
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
Sec	Second, Secondary
SF	Square Foot

Abbreviation	Meaning
SG	Specific Gravity
SLR	Solids Loading Rate
SMP	Stormwater Management Program, Shawnee Mission Park Pump Station
SND	Simultaneous Nitrification/ Denitrification
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station
SRT	Sludge Retention Time
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
TYP	Typical

Abbreviation	Meaning
U	
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VFA	Volatile Fatty Acids
VFAs	
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	Width
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation –Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WLWater LevelWK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

The purpose of this technical memorandum (TM) is to summarize the conceptual design of the disinfection facility at Mill Creek Regional (MCR) wastewater treatment plant (WWTP). This TM includes a discussion of available disinfection treatment technologies, disinfection treatment alternative evaluation, design criteria of the selected disinfection treatment technologies, footprint and layouts, capital costs, and operational and maintenance (O&M) costs.

For the disinfection treatment evaluation, a life-cycle cost analysis was developed. The conceptual cost opinion was developed as a 20-year present value (PV), which includes the effects of inflation, time-value of money, and equipment O&M. A triple bottom line (TBL) analysis was then completed as the basis for selection of the disinfection treatment alternatives for further consideration. Social, environmental, and operational criteria were weighted and scored to determine the benefit-cost of each alternative.

This TM is one in a series of technical memoranda for the MCR Facility Plan. Additional treatment processes and site optimization of these treatment facilities will be outlined in future TMs.

1.1 BACKGROUND

Prior to this Facility Plan for MCR, an extensive alternative analysis was done for the Tomahawk Creek (THC) WWTP Expansion. The results of this analysis can be used to inform the planning of the MCR Expansion. The THC WWTP is a good comparison because it is a similarly-sized facility (19 million gallons per day (mgd) annual average (AA) flow) with similar wastewater characteristics, is owned and operated by JCW, and has actual market costs for treatment technologies provided by a Contractor.

In August of 2014, Johnson County Wastewater (JCW) retained Black & Veatch (BV) for the project definition phase of the THC WWTP Expansion. The primary objective of the project definition phase was to confirm, through alternative development and evaluation, the optimal and proven treatment strategies throughout the WWTP for nutrient removal to meet current and anticipated future NPDES limits for design flows. Evaluation of these alternatives consisted of utilizing the Owner's TBL approach to evaluate non-economic factors in addition to developing capital and operating costs for each alternative. Each treatment process evaluation was presented to the Owner to select a recommended technology to be carried forward through design and construction.

After the project definition phase, the THC WWTP Expansion was continued into detailed design, followed by construction. The construction is scheduled to be completed in 2021. During the detailed design phase, some of the selected treatment technologies were re-evaluated and eventually revised as part of a value engineering effort. The treatment technologies that were part of the final design and eventually carried into construction serve as a valuable comparison for the MCR WWTP.

From TM 1 – Background, Flows, Loadings, and NPDES Permitting, the design flows for the MCR WWTP were established (as shown in Table 1-1). In order to ensure permit compliance, disinfection processes will be sized for peak day flows (126 mgd). During peak day conditions, all secondary effluent will be diverted from the filters to disinfection while auxiliary flows will be sent directly from the head of the plant to filtration followed by disinfection.

The disk filters described in TM 4 – Auxiliary Wet Weather Treatment play an important role in disinfection. The filters act as a barrier, removing total suspended solids (TSS), along with the associated bacteria and viruses attached to the TSS. Filtered effluent has a higher UV transmittance

(UVT) and less competing material for chlorine to react with. Final clarifier effluent has a low TSS concentration and a relatively high UVT, making it more suitable for diversion around filtration than the untreated influent associated with auxiliary flows. Data collected by Black & Veatch in other auxiliary treatment studies have shown filtered diluted influent flow will produce UVT similar to that of unfiltered secondary flow.

Table 1-1 MCR Design Flows

	DIURNAL LOW AA STARTUP[1]	AA STARTUP[2]	AA ULTIMATE	MAX MONTH	PEAK SECONDARY	PEAK DAY
MCR Design Flows, mgd	6.01	12.02	21.0	31.5	63.0	126.0

1 Historically this is 1/2 of the diurnal high (AA startup).
2 Flow projection based on Figure 2-3 (TM 1), year 2034 startup, assuming 1% growth.
1 Historically this is 1/2 of the diurnal high (AA startup).
2 Flow projection based on Figure 2-3 (TM 1), year 2034 startup, assuming 1% growth.

1.2 EXISTING ULTRAVIOLET DISINFECTION SYSTEM

All secondary wastewater effluent at MCR WWTP currently flows by gravity through an open channel ultraviolet (UV) disinfection facility before being discharged to the Kansas River. A summary of the existing UV system's design operating conditions is provided in Table 1-2.

Table 1-2 Existing UV System Design Operating Conditions

PARAMETER	DESIGN CRITERIA
Peak Day Flow, mgd	24
Maximum Month Flow, mgd	18
Average Annual Flow, mgd	12
Number of Channels	2
Flow per Channel, mgd	Minimum: 3 Average Daily: 6 Peak: 12
UV Transmittance	65%

The existing UV Disinfection Facility consists of two open channels, each containing a TrojanUV3000Plus™ low pressure high output UV disinfection system. A summary of the existing UV equipment is provided in Table 1-3.

Table 1-3 Existing UV Equipment Summary

PARAMETER	DESIGN CRITERIA
Model	Trojan Technologies, Inc. UV3000 Plus
Type	Low Pressure High Output
Number of Channels	2
Number of Banks per Channel	2
Number of Modules per Bank	7
Number of UV Lamps per Module	8
Minimum Number of Lamps	224
Maximum Total Power Consumption, kW	56

Installed in 2006, the existing UV system will be approaching 30 years of service when the MCR WWTP is expected to undergo expansion. The existing UV channels cannot be retrofitted to become compatible with state-of-the-art UV technology such as the TrojanUVSigna™ system described in Section 2.2. Due to the age of existing equipment and technology, limitations associated with retrofitting the existing structure, and site optimization, it is recommended that the existing UV Disinfection Facility be replaced during plant expansion.

1.3 VIRUS PERMIT LIMIT IMPACTS TO DISINFECTION

During the preliminary design phase of the THC WWTP, sodium hypochlorite was selected as the design mode of disinfection due the plant's design peak hour flow of 196 mgd and a seemingly imminent virus limit in NPDES permitting. USEPA was considering the conversion of a pathogen indicator-based disinfection requirement to a viral-based requirement. More than five years after the decision was made to pursue sodium hypochlorite disinfection at THC WWTP, USEPA has still not implemented a viral-based disinfection requirement. The regulatory community has been unable to agree upon how to develop viral permit limits, or what the "target" virus of such regulation might be.

Table 1-4 was developed based on information provided by the International Ultraviolet Association (IUVA) and various scientific white papers focused on chlorine treatment of viruses. The table shows the UV dose and free chlorine CT value required for a given log removal of various viruses.

The selection of a "target" virus and its respective removal is critical to the design of any disinfection system. The possibility of a virus limit does not rule out UV as a viable disinfection alternative. Table 1-4 shows that some viruses — such as *Adenovirus* and *JC Polyomavirus* — are resistant to UV light exposure, but can be adequately removed with a small dose of chlorine. It is recommended that UV remain as a viable disinfection alternative until KDHE develops either a water quality standard for virus, or incorporates virus standards into the use designation for the Kansas River.

Table 1-4 UV Dose and Free Chlorine CT Value for Given Log Removal of Various Viruses

NAME OF ORGANISM	UV DOSE FOR A GIVEN LOG REDUCTION WITHOUT PHOTOREACTIVATION (UV 254) MJ/CM2			CT FOR A GIVEN LOG REMOVAL (FREE CHLORINE) CT-MG-MIN/L		
	1	2	3	1	2	3
Adenovirus	35	69	103	0.02	0.06	0.15
Calicivirus Feline	7	15	22			1.5
Coxsackievirus	8	26	25	0.1 to 3.6	0.1 to 5.5	7.4
Echovirus	8	27	25	0.96	1.3	1.5
Hepatitis	6	10	15			
JC Polyomavirus	60	124	171			12
MS2	10	50	85		8-25	
Myoviridae	1.8	3.6	5.1			
Murine Norovirus	10	15	22	0.02	0.02	0.02
PHI X 174	3	5	7.5	30		
Picornaviridae aphthovirus	25	50	75	30		
Polivirus	8	16	23	30		1
Tectiviridae	10	17	24			
QB	8	18	28	30		
Reovirus	16	22		30		
Siphoviridae	1.8	3.6	5.7	30		
T1				30		
T4	4.3	8.5	13	30		
T7	2.9	6.9	14	30		
V1	3.1	5.9	8.8	30		

2.0 Design Criteria

A comparison of disinfection approaches has been developed to provide JCW with a range of technical and economic alternatives for future planning and design of the disinfection process. The alternatives considered viable for this project were based on the extensive evaluation completed as part of the THC WWTP design process. Using the information from that study in discussions with JCW, the following four alternatives were carried forward into evaluation for the MCR WWTP. Each alternative has been designed to achieve compliance with interim and final disinfection permit limits.

- Alternative 1 – Sodium Hypochlorite Disinfection
- Alternative 2 – UV Disinfection
- Alternative 3 – UV & Sodium Hypochlorite Disinfection
- Alternative 4 – Multi-Barrier Disinfection

2.1 ALTERNATIVE 1 - SODIUM HYPOCHLORITE DISINFECTION

Alternative 1 consists of the addition of sodium hypochlorite disinfection for the design peak day flow of 126 mgd. Figure 2-1 shows the Alternative 1 disinfection process schematic, including a visual representation of how flows would be split at different conditions.

Under this alternative, hypochlorite would be injected into each contact zone of a chlorine contact basin (CCB). The CCB under this alternative would consist of an influent distribution zone, five contact zones and an effluent channel. To comply with chlorine residual regulations, sodium bisulfite would be added to CCB effluent for dechlorination.

Each contact zone would be isolated by an electrically actuated slide gate. This allows for the use of portions of the basin for dry weather and smaller storm events without having to place all zones in service. Contact Zones 1-3 are each sized to handle 21 mgd of flow — totaling 63 mgd — peak secondary flow. Contact Zones 4-5 are each designed to handle up to 31.5 mgd of auxiliary flow.

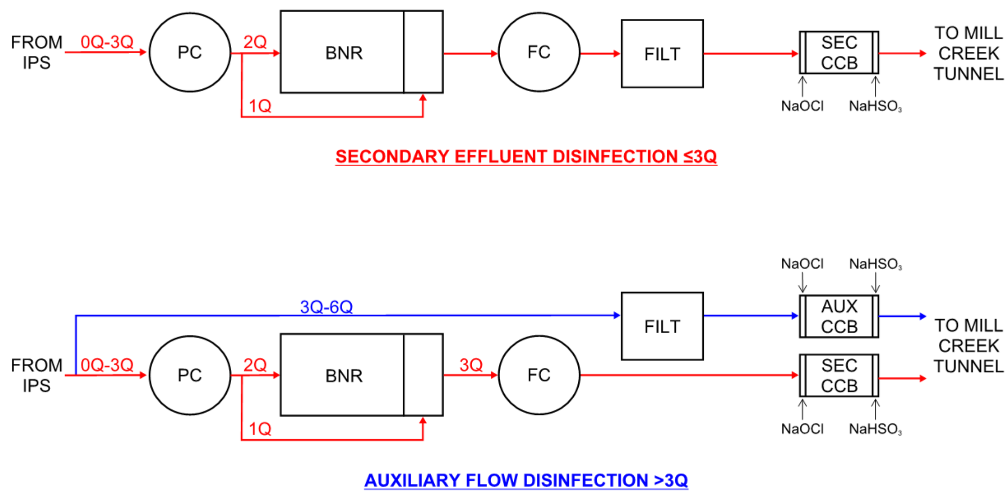


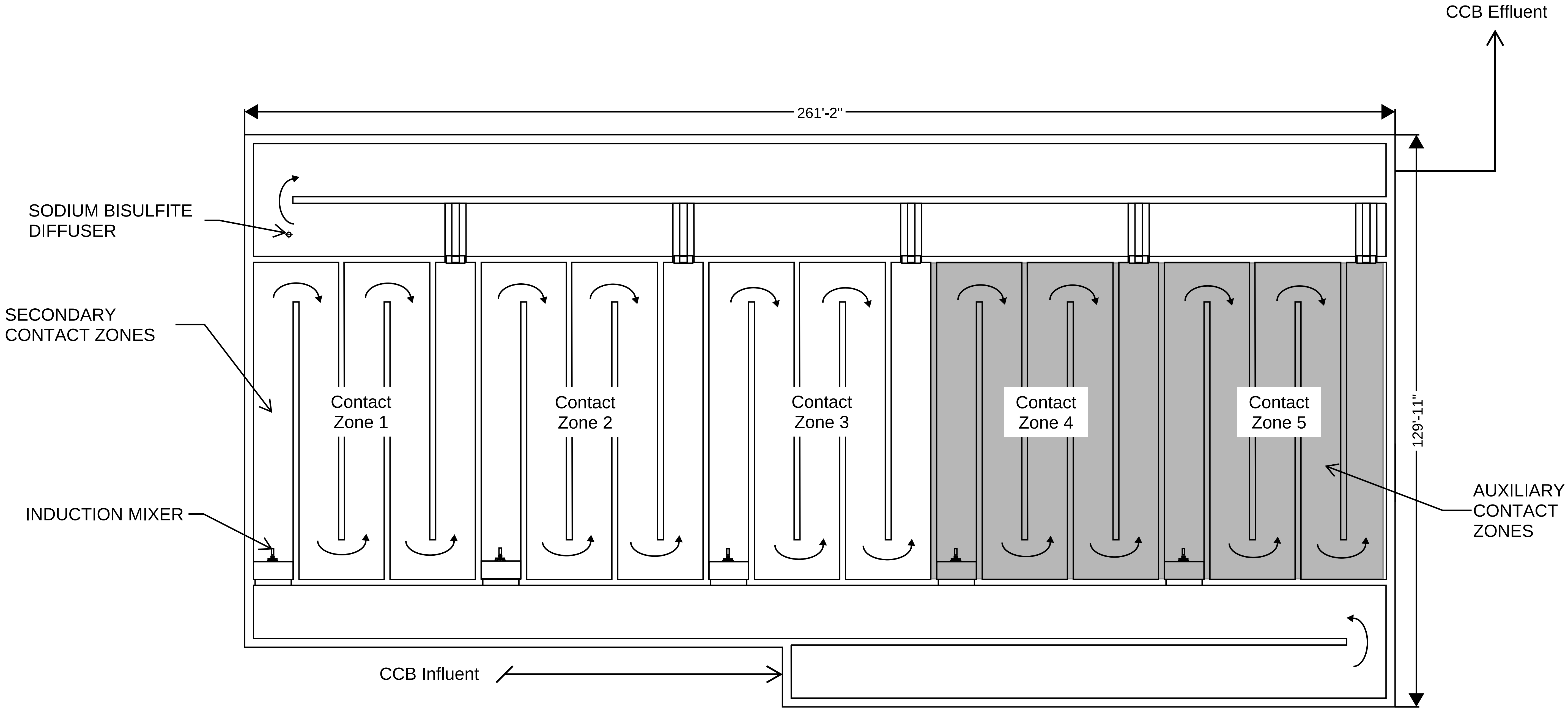
Figure 2-1 Alternative 1 - Sodium Hypochlorite Disinfection Schematic

It should be noted that secondary flows under Alternatives 1 and 4 would utilize "free chlorine" disinfection while the auxiliary flows under Alternatives 1, 3 and 4 would receive "combined chlorine and chloramine" disinfection. Free chlorine disinfection can be characterized by a higher initial chlorine dose and a significant loss of chlorine to side reactions or chlorine demand. Chloramine disinfection uses ammonia in the waste stream, and the free chlorine reacts with ammonia to form chloramine instantaneously resulting in less chlorine demand consumption. Chloramine is a good disinfectant, but a very weak oxidizer; it is easier to maintain an effluent chlorine residual. Free chlorine is highly reactive, and, at longer retention times, has difficulties maintaining target chlorine residual. Hypochlorite dosing into the secondary and auxiliary treatment contact zones will, therefore, be different.

Facilities associated with sodium hypochlorite disinfection include a CCB and disinfection chemical building (DCB). Shown in Figure 2-2, the CCB under this alternative will be divided into five contact zones. A hypo induction mixing system will be installed at the head of each contact zone. The CCB effluent channel will be equipped with a sodium bisulfite diffuser for dechlorination. A control system will be provided that adjusts the hypochlorite and bisulfite application to each train. The design of each train is based on recommendations provided in the 2006 USEPA Wastewater Disinfection Guidance Manual with the width to depth ratio being 1:1 and the length to depth ratio being greater than 40:1. The secondary chlorine contact basins were designed for 15 minutes of contact at peak secondary flow while the auxiliary flow basins were designed for 10 minutes of contact time at during peak auxiliary flow conditions. Black & Veatch has developed 10 minutes of contact during auxiliary flow conditions based on empirical testing at a number of other facilities. Design criteria for the CCB are summarized in Table 2-1.

Table 2-1 Alternative 1 - CCB Design Criteria

PARAMETER	DESIGN CRITERIA
Contact Zone Quantity	
Secondary	3
Auxiliary	2
Contact Zone Capacity, EA, mgd	
Secondary @ 15 min CT	21
Auxiliary @ 10 min CT	31.5
Contact Zone Dimensions (Secondary/Auxiliary)	
Passes per Contact Zone	5
Pass Width, ft	9
Pass Length, ft	72
Side Water Depth, ft	9
Sodium Hypochlorite Induction Mixers	
Mixer Quantity	5
Motor Rating, HP, each	10
Sodium Bisulfite Diffuser Quantity	1



SCALE 1/24" = 1'-0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

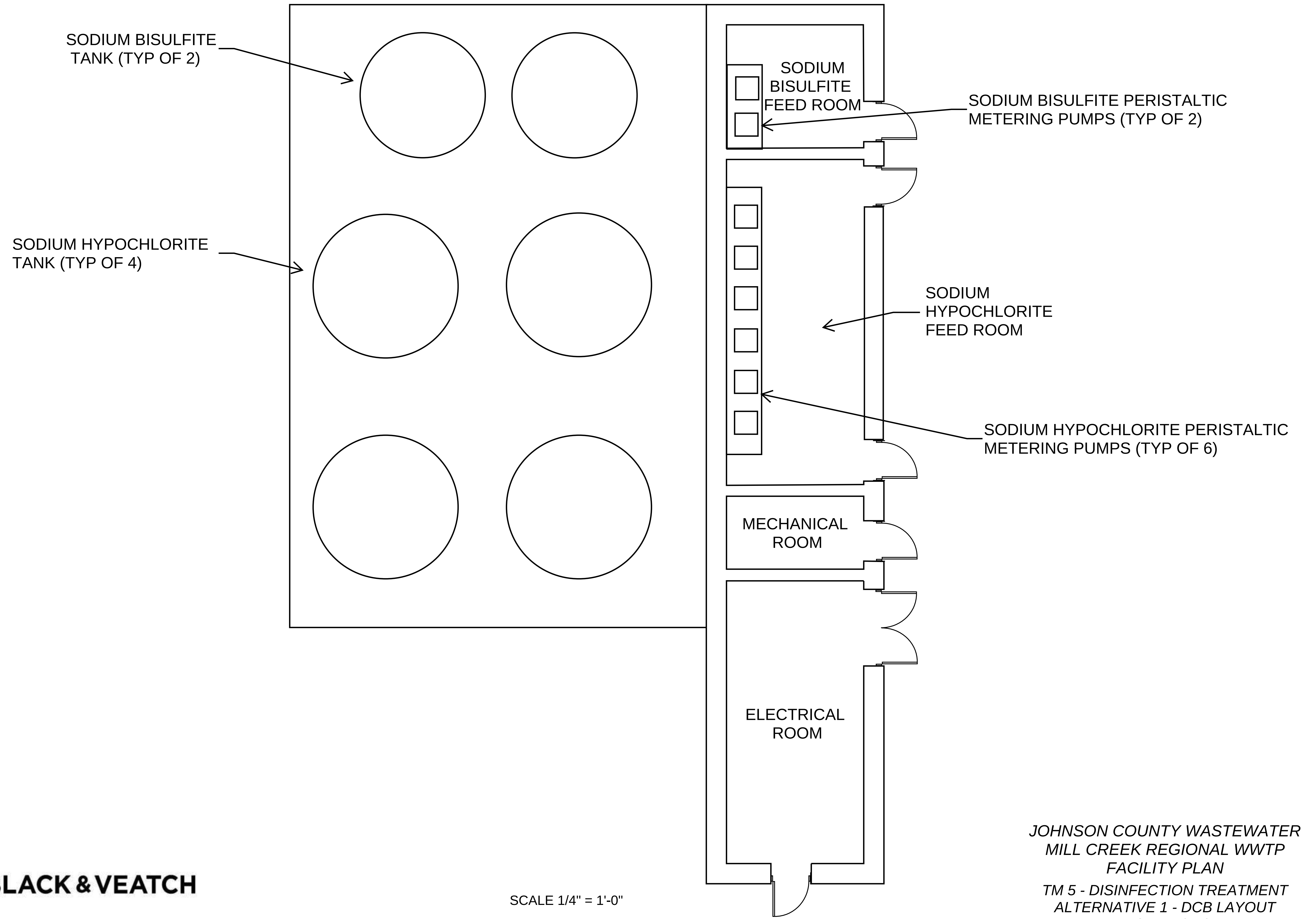
TM 5 - DISINFECTION TREATMENT
ALTERNATIVE 1 - CCB LAYOUT
FIGURE 2 - 2

The DCB shown in Figure 2-3 will be located near the CCB. The at-grade masonry structure will consist of an electrical room, mechanical room, and two chemical feed rooms. Sodium hypochlorite and sodium bisulfite will be stored outside of the structure in double contained polyethylene chemical storage tanks. Chemical storage tank sizing was based on the size provided at THC WWTP. In total, there will be four (4) 8,700-gallon sodium hypochlorite tanks and two (2) 6,500-gallon sodium bisulfite tanks. A total of eight peristaltic metering pumps will be located inside the DCB, six pumps (five duty, one standby) dedicated to sodium hypochlorite and two pumps (one duty, one standby) dedicated to sodium bisulfite. The design criteria associated with the DCB are summarized in Table 2-2.

Table 2-2 Alternative 1 - DCB Design Criteria

PARAMETER	DESIGN CRITERIA
Sodium Hypochlorite Solution Strength	12.5%
Avg. Chlorine Dose, mg/L Cl_2	
Secondary	6.0
Auxiliary	10.0
Total Sodium Hypochlorite Usage, gpy	371,500
Sodium Bisulfite Solution Strength	40%
Avg. Chlorine Residual, mg/L	
Secondary	2.0
Auxiliary	4.0
Total Sodium Bisulfite Usage, gpy	49,600
Chemical Metering Pumps	
Pump Type	Peristaltic
Pump Quantity	
Sodium Hypochlorite	6 (5 Duty, 1 Standby)
Sodium Bisulfite	2 (1 Duty, 1 Standby)
Flow Range, gph	
Sodium Hypochlorite	6.5 - 360.0
Sodium Bisulfite	1.0 - 100.0
Chemical Storage Tanks	
Tank Type	Double Contained Polyethylene
Tank Capacity, gal	

PARAMETER	DESIGN CRITERIA
Sodium Hypochlorite	8,700
Sodium Bisulfite	6,500
Tank Quantity	
Sodium Hypochlorite	4
Sodium Bisulfite	2
Combined Storage Time at AA Flow, days	
Sodium Hypochlorite	39
Sodium Bisulfite	111
Combined Storage Time at Peak Flow, days	
Sodium Hypochlorite	8
Sodium Bisulfite	18



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JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM 5 - DISINFECTION TREATMENT
ALTERNATIVE 1 - DCB LAYOUT
FIGURE 2 - 3

2.2 ALTERNATIVE 2 - UV DISINFECTION

Alternative 2 consists of UV disinfection for all flows up to 126 mgd. As noted in Section 1.2, MCR WWTP currently utilizes TrojanUV3000Plus™ technology to disinfect plant flows. At the time of this evaluation, Trojan Technologies has shifted research and development efforts away from the TrojanUV3000Plus™ system toward Trojan's new standard technology, TrojanUVSigna™. Trojan will continue to service existing TrojanUV3000Plus™ systems, but efforts to improve technology have shifted to TrojanUVSigna™.

The TrojanUV Solo Lamp™ standard to TrojanUVSigna™ is a 1,000-Watt lamp that combines features of low and medium pressure lamps to optimize disinfection performance while minimizing maintenance. Fewer lamps with a longer lifespan provide for reduced operations and maintenance costs in comparison with a TrojanUV3000Plus™ system. Another advantage of the TrojanUVSigna™ is that it does not require a lifting mechanism for maintenance. General maintenance, such as lamp changing, can be performed while banks are in the disinfection channel. An integral Automatic Raising Mechanism (ARM) simplifies other, more detailed maintenance to the TrojanUVSigna™ system or the disinfection channel.

For evaluation purposes, it is assumed that a TrojanUVSigna™ system will be the selected mode of technology carried forward for evaluation under Alternatives 2, 3 and 4. At the time of the design, the UV technology will need to be reviewed, and, if appropriate, the newest technology incorporated into the future system.

The key design parameter for the design of the UV system is transmittance. The design transmittance for both the secondary and auxiliary treatment was based on 65 percent. This allows for all the channels to be used during both wet and dry conditions providing flexibility to operations staff. Typical operating range for a TrojanUVSigna™ system is between 20 percent and 100 percent output, providing operational flexibility under varying flow conditions; however, it should be noted that, if flows reach a point requiring less than 20 percent output from the UV system, channels can be removed from service to bring the still operating TrojanUVSigna™ systems back into an acceptable range of operation.

Since the water quality of auxiliary and secondary flows are approximately the same post auxiliary filtration, particularly for UV transmittance, the system can have the same equipment in all channels to provide flexibility and redundancy.

Refer to Figure 2-4 for a schematic of secondary and auxiliary flows under Alternative 2.

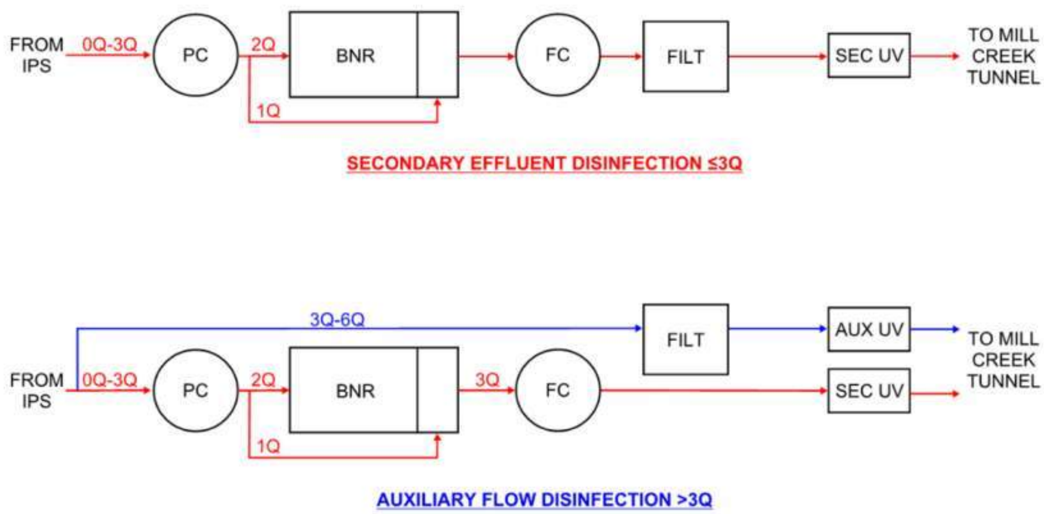


Figure 2-4 Alternative 2 - UV Disinfection Schematic

As shown in Figure 2-5, the UV Building under this alternative consists of a single-story masonry structure with an influent channel, 6 disinfection channels (each capable of treating up to 21 mgd), an effluent channel, and an electrical room.

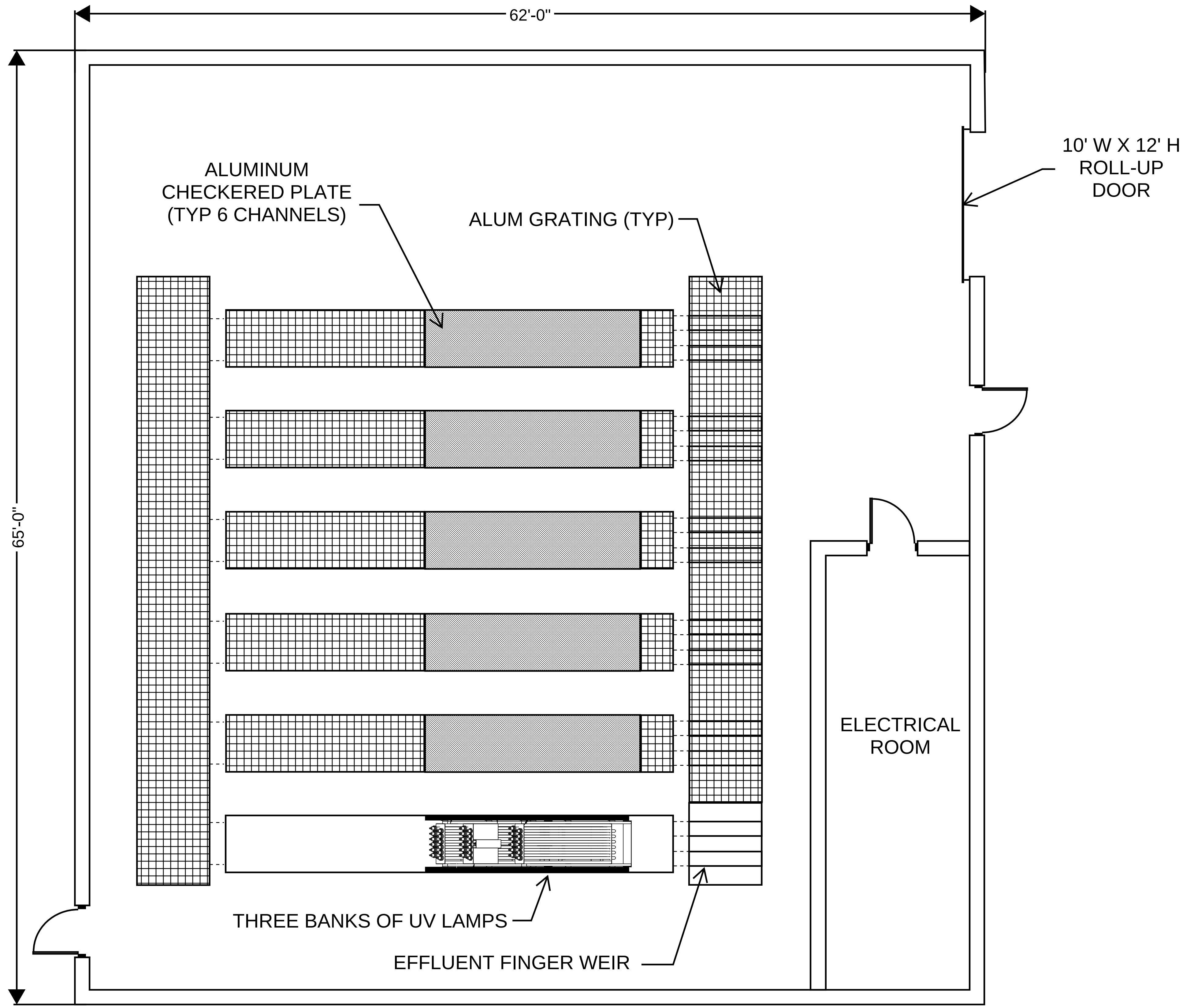
The design criteria associated with the Alternative 2 UV Building are summarized in Table 2-3.

Table 2-3 Alternative 2 - UV Building Design Criteria

PARAMETER	DESIGN CRITERIA
No. Channels	
Secondary	3
Auxiliary	3
Channel Capacity, EA, mgd	
Secondary	21
Auxiliary	21
Channel Dimensions	
Min. Channel Length, ft.	30.6
Channel Width at Banks, ft.	4
Channel Depth, ft.	7.8
Number of Banks per Channel	3
Number of UV Lamps per Bank	20
Total Number of UV Lamps	360

PARAMETER	DESIGN CRITERIA
Min. UV Transmittance (1)	65%
Design Dose, mJ/cm^2	40
Avg. Power Draw, kW/h	63
Max. Power Draw, kW/h	379

[1] Testing completed by Black & Veatch has shown that cloth media filtration on auxiliary flows has produced 65 percent transmittance water.



SCALE 1/8" = 1'-0"

BLACK & VEATCH

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM 5 - DISINFECTION TREATMENT
ALTERNATIVE 2 - UV BUILDING LAYOUT
FIGURE 2 - 5

2.3 ALTERNATIVE 3 – UV & SODIUM HYPOCHLORITE DISINFECTION

Alternative 3 serves as a combination of Alternatives 1 and 2. Shown in Figure 2-6, secondary flows would be directed to a UV building containing three disinfection channels, each utilizing TrojanUVSigna™ technology to treat up to 21 mgd (total of 63 mgd). As described in Section 1.1, during auxiliary flow conditions above 63 mgd, secondary flows will be diverted around filtration before receiving disinfection. Auxiliary flows under Alternative 3 would be routed from the head of the plant to filtration before receiving sodium hypochlorite disinfection. The CCB would consist of 2 contact zones each sized to treat 31.5 mgd, matching the auxiliary flow contact zones under Alternative 1.

The challenge of this alternative is managing the degradation of the hypochlorite during extended periods of time in which the auxiliary disinfection process is not utilized. The benefit is that hypochlorite would only be utilized for auxiliary flows, reducing the operational cost.

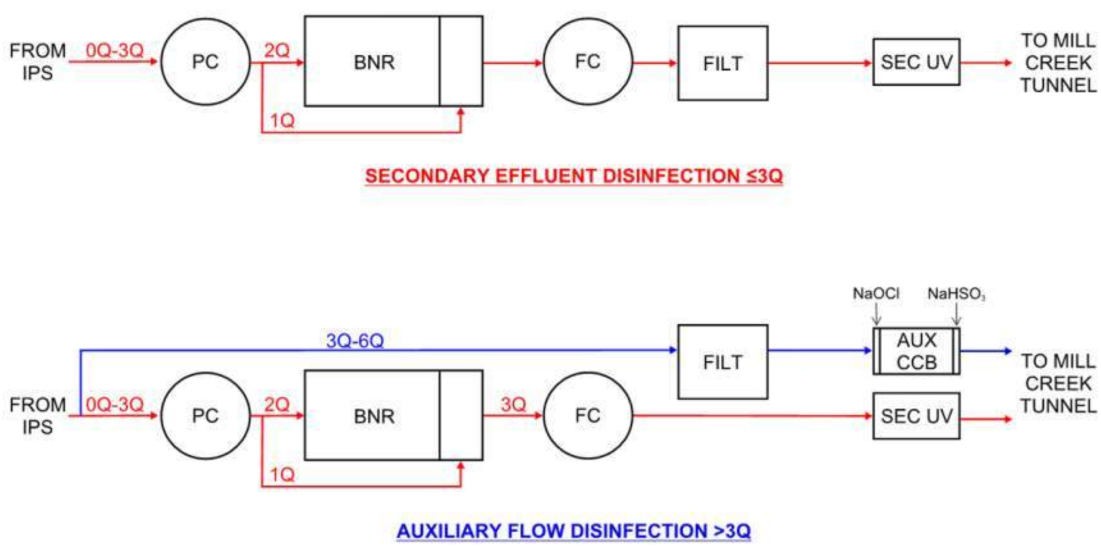


Figure 2-6 Alternative 3 - UV & Sodium Hypochlorite Disinfection

The UV Building under this alternative will consist of a single-story masonry structure with three UV channels and an electrical room, similar to the UV building shown in Figure 2-5. The design criteria associated with the Alternative 3 UV Building are summarized in Table 2-4.

Table 2-4 Alternative 3 - UV Building Design Criteria

PARAMETER	DESIGN CRITERIA
No. Channels	
Secondary	3
Channel Capacity, EA, mgd	
Secondary	21
Channel Dimensions	
Min. Channel Length, ft.	30.6

PARAMETER	DESIGN CRITERIA
Channel Width at Banks, ft.	4
Channel Depth, ft.	7.8
Number of Banks per Channel	3
Number of UV Lamps per Bank	20
Total Number of UV Lamps	180
Min. UV Transmittance	65%
Design Dose, mJ/cm^2	40
Avg. Duty Power Draw, kWh	32
Max. Power Draw, kW/h	189.5

Facilities associated with sodium hypochlorite disinfection under Alternative 3 include a CCB and DCB. The CCB will be divided into two contact zones. A sodium hypochlorite induction mixing system will be installed at the head of each contact zone. The effluent channel will be equipped with a sodium bisulfite diffuser. Design criteria for the Alternative 3 CCB are summarized in Table 2-5.

Table 2-5 Alternative 3 - CCB Design Criteria

PARAMETER	DESIGN CRITERIA
Contact Zone Quantity	
Auxiliary	2
Contact Zone Capacity, EA, mgd	
Auxiliary	31.5
Contact Zone Dimensions	
Passes per Contact Zone	5
Pass Width, ft	9
Pass Length, ft	72
Side Water Depth, ft	9
Sodium Hypochlorite Induction Mixers	
Mixer Quantity	2
Motor Rating, HP, each	10
Sodium Bisulfite Diffuser Quantity	1
Slide Gate Quantity	2

The associated DCB will be located near the CCB. The at-grade masonry structure will consist of an electrical room, mechanical room, and two chemical feed rooms. Sodium hypochlorite and sodium bisulfite will be stored outside of the structure in double contained polyethylene chemical storage tanks. In total, there will be two (2) 8,700-gallon sodium hypochlorite tanks and one (1) 6,500-gallon sodium bisulfite tank. A total of five peristaltic metering pumps will be located inside the DCB, three pumps (two duty, one standby) dedicated to sodium hypochlorite and two pumps (one duty, one standby) dedicated to sodium bisulfite. The design criteria associated with the DCB are summarized in Table 2-6.

Table 2-6 Alternative 3 - DCB Design Criteria

PARAMETER	DESIGN CRITERIA
Sodium Hypochlorite Solution Strength	12.5%
Avg. Chlorine Dose, mg/L Cl_2	
Auxiliary	10.0
Total Sodium Hypochlorite Usage, gpy	8,850
Sodium Bisulfite Solution Strength	40%
Avg. Chlorine Residual, mg/L	
Auxiliary	4.0
Total Sodium Bisulfite Usage, gpy	1,400
Chemical Metering Pumps	
Pump Type	Peristaltic
Pump Quantity	
Sodium Hypochlorite	3 (2 Duty, 1 Standby)
Sodium Bisulfite	2 (1 Duty, 1 Standby)
Flow Range, gph	
Sodium Hypochlorite	6.5 - 360.0
Sodium Bisulfite	1.0 - 100.0
Chemical Storage Tanks	
Tank Type	Double Contained Polyethylene
Tank Capacity, gal	
Sodium Hypochlorite	8,700
Sodium Bisulfite	6,500
Tank Quantity	

PARAMETER	DESIGN CRITERIA
Sodium Hypochlorite	2
Sodium Bisulfite	1
Combined Storage Time at Peak Flow, days	
Sodium Hypochlorite	4
Sodium Bisulfite	9

2.4 ALTERNATIVE 4 – MULTI-BARRIER DISINFECTION

Multi-barrier disinfection as defined for Alternative 4 would also be a combination of Alternatives 1 and 2 with the addition of a dose of chlorine upstream of the UV process. The overall goal of this approach is to provide a greater microbial and virus removal barrier. During normal operation, sodium hypochlorite would be injected into final clarifier effluent upstream of the disk filters followed by UV disinfection before exiting the plant. Secondary flows will be diverted around filtration before receiving UV disinfection when plant flows exceed 63 mgd. Auxiliary flows under Alternative 4 would be routed from the head of the plant to filtration before receiving sodium hypochlorite disinfection. The CCB would consist of 2 contact zones each sized to treat 31.5 mgd of flows, matching the auxiliary flow contact zones under Alternatives 1 and 3. Refer to Figure 2-7 for a schematic of Alternative 4.

The advantage of this alternative is that it provides a multibarrier approach for being able to remove more challenging viruses or to remove microbial organisms that have not already been identified. This approach incorporates the disinfection barriers that both chlorine and UV provide, which is similar to what would be observed at either a potable or reuse treatment facility.

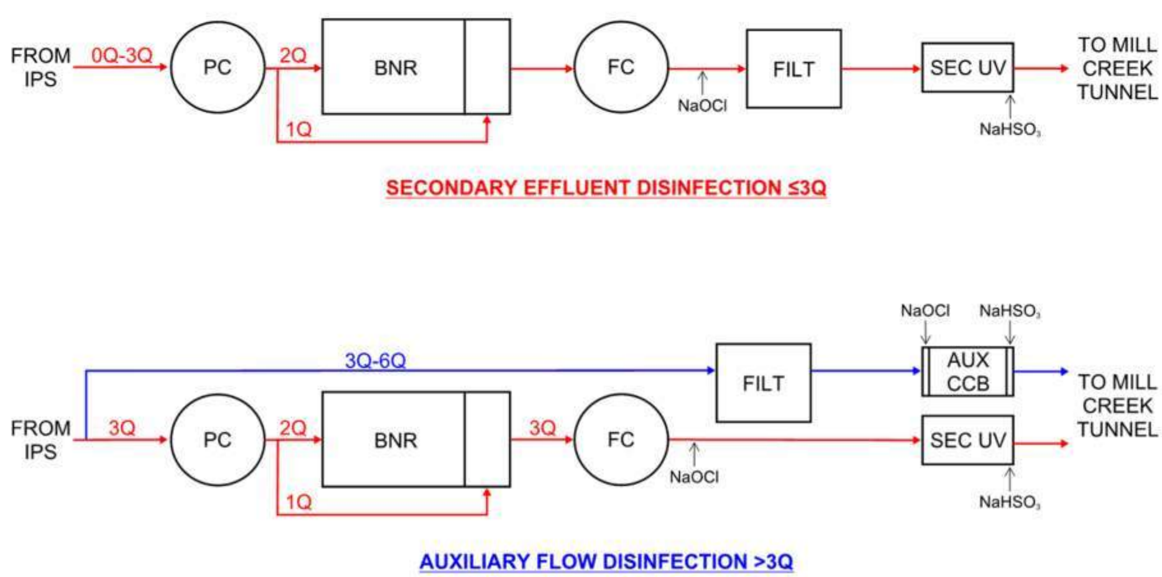


Figure 2-7 Alternative 4 - Multi-Barrier Disinfection Schematic

The UV Building under this alternative will consist of a single-story masonry structure with three UV channels and an electrical room, similar to the UV building shown in Figure 2-5. Sodium hypochlorite will be injected upstream of the disk filters to provide adequate contact time before flows reach the UV building, as well as for maintenance purposes to control algae growth in the filters. A single sodium bisulfite diffuser will be located in the combined UV effluent channel. The design criteria associated with the Alternative 4 UV Building are summarized in Table 2-7.

Table 2-7 Alternative 4 - UV Building Design Criteria

PARAMETER	DESIGN CRITERIA
No. Channels	
Secondary	3
Channel Capacity, EA, mgd	
Secondary	21
Channel Dimensions	
Min. Channel Length, ft.	30.6
Channel Width at Banks, ft.	4
Channel Depth, ft.	7.8
Number of Banks per Channel	3
Number of UV Lamps per Bank	20
Total Number of UV Lamps	180
Min. UV Transmittance	65%
Design Dose, mJ/cm^2	40
Avg. Duty Power Draw, kWh	32
Max. Power Draw, kW/h	189.5
Sodium Hypochlorite Induction Mixers[1]	
Mixer Quantity	1
Motor Rating, HP	10
Sodium Bisulfite Diffuser Quantity	1
[1] Induction mixer located upstream of disk filters.	

Facilities associated with sodium hypochlorite disinfection under Alternative 4 include a CCB and DCB. The CCB will be divided into two contact zones. A sodium hypochlorite induction mixing system will be installed at the head of each contact zone. The effluent channel will be equipped with a sodium bisulfite diffuser. Design criteria for the Alternative 4 CCB are summarized in Table 2-8.

Table 2-8 Alternative 4 - CCB Design Criteria

PARAMETER	DESIGN CRITERIA
Contact Zone Quantity	
Auxiliary	2
Contact Zone Capacity, EA, mgd	
Auxiliary	31.5
Contact Zone Dimensions	
Passes per Contact Zone	5
Pass Width, ft	9
Pass Length, ft	72
Side Water Depth, ft	9
Sodium Hypochlorite Induction Mixers	
Mixer Quantity	2
Motor Rating, HP	10
Sodium Bisulfite Diffuser Quantity	1
Slide Gate Quantity	2

The associated DCB will be located near the CCB. The at-grade masonry structure will consist of an electrical room, mechanical room, and two chemical feed rooms. Sodium hypochlorite and sodium bisulfite will be stored outside of the structure in double contained polyethylene chemical storage tanks. In total, there will be three (3) 8,700-gallon sodium hypochlorite tanks and one (1) 6,500-gallon sodium bisulfite tank. A total of seven peristaltic metering pumps will be located inside the DCB, four pumps (two duty, two standby) dedicated to sodium hypochlorite and three pumps (two duty, one standby) dedicated to sodium bisulfite. The design criteria associated with the DCB are summarized in Table 2-9.

Table 2-9 Alternative 4 - DCB Design Criteria

PARAMETER	DESIGN CRITERIA
Sodium Hypochlorite Solution Strength	12.5%
Avg. Chlorine Dose, mg/L Cl_2	
Secondary	3.0
Auxiliary	10.0
Total Sodium Hypochlorite Usage, gpy	190,200

PARAMETER	DESIGN CRITERIA
Sodium Bisulfite Solution Strength	40%
Avg. Chlorine Residual, mg/L	
Secondary	0.5
Auxiliary	4.0
Total Sodium Bisulfite Usage, gpy	13,000
Chemical Metering Pumps	
Pump Type	Peristaltic
Pump Quantity	
Sodium Hypochlorite	4 (2 Duty, 2 Standby)
Sodium Bisulfite	3 (2 Duty, 1 Standby)
Flow Range, gph	
Sodium Hypochlorite	6.5 - 360.0
Sodium Bisulfite	1.0 - 100.0
Chemical Storage Tanks	
Tank Type	Double Contained Polyethylene
Tank Capacity, gal	
Sodium Hypochlorite	8,700
Sodium Bisulfite	6,500
Tank Quantity	
Sodium Hypochlorite	3
Sodium Bisulfite	1
Combined Storage Time at AA Flow, days	
Sodium Hypochlorite	59
Sodium Bisulfite	221
Combined Storage Time at Peak Flow, days	
Sodium Hypochlorite	6
Sodium Bisulfite	9

3.0 Cost Analysis

Preliminary Capital and O&M costs were developed for each of the disinfection alternatives described in Section 2.0. The basis of design presented in this TM was used to develop a preliminary opinion of probable construction cost for each disinfection alternative. The costs presented below do not include cost of electrical, sitework, instrumentation and control (I&C), engineering, legal, administrative (ELA), or contingencies. These costs will appear as line items in the overall opinion of probable construction cost presented in the Facility Plan Report. This costing method allows for a direct comparison of costs for each alternative.

3.1 SUMMARY OF CAPITAL COSTS

The capital costs associated with each alternative are summarized in Table 3-1.

Table 3-1 Disinfection Treatment Alternatives Capital Cost Summary

ALTERNATIVE	EQUIPMENT CAPITAL COST	TOTAL CAPITAL COST
Alternative 1 - Sodium Hypochlorite Disinfection	$ 926,000	$ 7,689,000
Alternative 2 - UV Disinfection	$ 4,180,000	$ 6,121,000
Alternative 3 - UV & Sodium Hypochlorite Disinfection	$ 2,585,000	$ 7,292,000
Alternative 4 - Multi-Barrier Disinfection	$ 2,772,000	$ 7,825,000

- Capital costs presented in 2020 dollars.
- Costs exclude electrical, site, I&C, ELA and contingencies.
- Capital costs are conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

The CCB associated with Alternatives 1, 3, and 4 was modeled after the CCB at the THC WWTP. Adjustments to the structure were made to accommodate lower flows and a larger site than designed for at THC WWTP. Associated DCBs are modeled after the DCB at THC WWTP, with the exception of the chemical storage tanks (which were sized and priced independently). The layout of the UV Building in Alternatives 2, 3, and 4 was modeled after the Marcy Gulch WWTP in Centennial, Colorado, a recent BV project with a similarly-sized UV Building also housing TrojanUVSigna™ technology. UV system quotes were given by Trojan UV while the remainder of the structure costs were obtained by applying a unit price per square foot, calculated from buildings of similar complexity at the THC WWTP.

Alternative 2 has the lowest associated capital cost. This is primarily due to the high capital costs associated with constructing large concrete basins required for sodium hypochlorite disinfection. Concrete quantities for the CCB's in Alternatives 1, 3, and 4 range from 1,700 to 3,900 cubic yards. Structure costs associated with UV disinfection are relatively low when compared to the price of constructing a CCB and its associated DCB. The TrojanUVSigna™ equipment accounts for around 50 percent of the capital cost of the UV portions of Alternatives 2, 3, and 4.

3.2 SUMMARY OF OPERATION AND MAINTENANCE COSTS

O&M costs include the cost of power, chemicals, operating labor and general maintenance. The O&M costs associated with each alternative are summarized in Table 3-2. The estimates are in 2020 dollars.

Table 3-2 Disinfection Treatment Alternatives O&M Annual Cost Summary

	ALTERNATIVE 1 - SODIUM HYPOCHLORITE DISINFECTION	ALTERNATIVE 2 - UV DISINFECTION	ALTERNATIVE 3 - UV & SODIUM HYPOCHLORITE DISINFECTION	ALTERNATIVE 4 - MULTI-BARRIER DISINFECTION
Power	$ 3,900	$ 23,000	$ 22,400	$ 26,400
Labor	$ 13,000	$ 18,000	$ 18,000	$ 26,000
Equipment Maintenance	$ 9,900	$ 55,700	$ 33,400	$ 41,200
Chemicals	$ 385,700	$ -	$ 9,600	$ 175,700
Total	**$ 412,500**	**$ 96,700**	**$ 83,400**	**$ 269,300**

Alternative 3 has the lowest associated O&M cost. Annual power costs were calculated by applying a rate of $0.073/kW to the annual power consumption calculated for the equipment in each alternative. Power costs associated with operating a sodium hypochlorite disinfection system were found to be far less than those associated with UV disinfection. The annual labor costs associated with each alternative were calculated by estimating the number of operators and frequency of maintenance expected for each system. A rate of $33.94 was then applied to the estimated hours in order to obtain the O&M costs presented in . Equipment maintenance was calculated as 2 percent of total equipment capital costs in an attempt to estimate replacement costs of equipment parts, lamps, ballasts, sleeves, and wipers. The results of the analysis indicate that the biggest differentiator in calculating O&M costs of the alternatives was the chemical cost of sodium hypochlorite. In order to estimate chemical consumption associated with Alternatives 1, 3, and 4, plant flow data between 2015 and 2019 was analyzed and translated into a design flow, providing an estimated number of days at different flow rates. Calculating this design flow allows for a more accurate estimate of total gallons of chemical required annually for each alternative. The continuous addition of sodium hypochlorite associated with Alternatives 1 and 4 is more expensive when compared to the intermittent feed Alternative 3.

3.3 PRESENT VALUE

The 20-year present value (PV) for each of the disinfection alternatives is summarized in Table 3-3. PV estimates are based on the following additional assumptions:

- Cost year basis: 2020
- Nominal Discount Rate: 3.10 percent
- Inflation Rate: 1.90 percent
- Resulting Net Discount Rate: 1.20 percent

To calculate the total O&M cost over the 20-year life cycle, the annual O&M cost for each year is calculated by multiplying the previous year's annual O&M cost by the inflation rate. That annual O&M cost for that specific year is then corrected back to 2020 dollars, and the nominal discount rate is applied. The sum of all the annual present values is the overall present value O&M cost over 20-years.

Table 3-3 Capital, O&M, and PV Cost Estimates (2020 $'s)

DESCRIPTION	CAPITAL COST	ANNUAL O&M COST	O&M PV (20 YEARS)	TOTAL PV
Alternative 1 – Sodium Hypochlorite Disinfection	$7,688,000	$412,500	$7,312,000	$15,001,000
Alternative 2 – UV Disinfection	$6,121,000	$96,700	$1,714,000	$7,122,000
Alternative 3 – UV & Sodium Hypochlorite Disinfection	$7,292,000	$83,400	$1,478,000	$8,773,000
Alternative 4 – Multi-Barrier Disinfection	$7,825,000	$269,300	$4,774,000	$12,218,000

Although capital costs were relatively close between the alternatives, large differences in annual O&M costs greatly impacted the PV evaluation. High chemical costs associated with Alternatives 1 and 4 resulted in these two alternatives having the highest present value. Alternative 3 only utilizes chemicals during storm events with flows exceeding 63 mgd, leading to less of a chemical consumption impact on annual O&M costs. Alternative 2 has the lowest PV by approximately $1,650,000 when compared to the alternative with the next lowest PV, making this the most cost effective alternative.

It is important to note that salvage value has not been included in this PV evaluation. Although there would be a salvage value for concrete in the 20-year PV life cycle for each of these alternatives, it is estimated that the similarity between structures would result in an across the board increase of similar magnitude for all alternatives. Since the goal of the total PV is to differentiate alternatives, salvage value has not been included.

3.4 TRIPLE BOTTOM LINE ANALYSIS

The disinfection alternatives were compared through triple bottom line (TBL) analysis. By factoring social and environmental considerations into the analysis along with economic information expressed as PV, a more thorough comparison of alternatives can be achieved. The benefit score was then combined with the PV to determine the benefit-cost of each alternative. The TBL criteria below in Table 3-4 were developed with JCW to capture MCR specific concerns as well as consistency with similar past evaluations.

Table 3-4 Evaluation Criteria and Descriptions

CRITERIA	DESCRIPTION
Flexibility / Turndown	Is alternative flexible enough to successfully adjust to changing conditions (i.e. flow and load)? How much can be treated through the process?
Performance Reliability	Are there adjustable controls, process options, and/or equipment features available for operators to respond to an upset? Is alternative resistant to an upset, and what are the consequences if upset does occur? Is the alternative a proven technology?
Operational Complexity / Maintenance	How complex is the alternative to operate, control and maintain? Does the alternative rely on more system components operating together? Are there major scheduled replacements and cleanings?
Layout / Constructability	How easily and cost-effectively can the alternative be phased to meet the start-up and construction constraints? How well does the alternative fit on the site? Do the facilities lay out in an orderly fashion (e.g., do trucks have to drive to through several facilities in order to access their final destination)?
Social Impacts	How well does the alternative prevent off-site impacts to public perception — such as truck traffic, noise, odor, visual aesthetics, etc. — and can these impacts be easily mitigated? (Impacts from construction activities are excluded.)
Environmental Impacts	How well does the alternative minimize the impact to the environment in terms of carbon footprint (during construction and use phase), ecosystem quality, and resource use?
Safety	How well does the alternative minimize safety risks to the plant staff and the public and can the risks be mitigated?
Regulatory Risk	How difficult will it be to obtain EPA and KDHE regulatory acceptance of the alternative? Could alternative acceptance be achieved in desired schedule?

Table 3-5 is a summary of the weighted scores for the Disinfection Alternatives. A ranking of 5 means this is the most important, or has the most positive impact. A ranking of 1 means the is the least important, or has the most negative impact.

Table 3-5 Disinfection Alternatives Triple Bottom Line Scoring

CRITERIA	RELATIVE WEIGHT	ALTERNATIVE 1 – SODIUM HYPOCHLORITE DISINFECTION		ALTERNATIVE 2 – UV DISINFECTION		ALTERNATIVE 3 – UV & SODIUM HYPOCHLORITE DISINFECTION		ALTERNATIVE 4 – MULTI-BARRIER DISINFECTION	
		Ranking	Weighted Score	Ranking	Weighted Score	Ranking	Weighted Score	Ranking	Weighted Score
Flexibility / Turndown	15%	3	4.5	3	4.5	3	4.5	3	4.5
Performance Reliability	20%	5	10	4	8	3	6	5	10
Operational Complexity / Maintenance	20%	4	8	5	10	3	6	3	6
Layout / Constructability	10%	3	3	4	4	4	4	4	4
Social Impacts	10%	2	2	4	4	4	4	2	2
Environmental Impacts	10%	2	2	5	5	4	4	3	3
Safety	10%	2	2	4	4	4	4	3	3
Regulatory Risk	5%	5	2.5	2	1	3	1.5	4	2
Total Weighted Score	**100%**	**34**		**40.5**		**34**		**34.5**	

Note: Rankings: 5 = Most Important or most positive impact. 1 = Least Important or most negative impact.

Results of the TBL analysis are favorable to Alternative 2. Not coincidentally, this is the only alternative that does not have continuous or intermittent use of chemicals in disinfecting flows. Categories impacted by chemical usage include social impacts, environmental impacts, and safety. Larger impacts were observed for alternatives with continuous chemical feed (Alternatives 1 and 4) than alternatives that had intermittent or no chemical feed. Alternative 3, which only has intermittent chemical feed, was rated lower based on Performance Reliability due to extended periods of time that the auxiliary basin is likely to experience without being put into service.

3.5 COST/BENEFIT SCORING

The sum of the TBL scoring can be converted to the normalized benefit score based upon the highest scoring alternative. The benefit score for each alternative is then divided into the respective PV to express the benefit score in economic terms. Table 3-6 contains the PV to the normalized benefit ratio for each alternative.

Table 3-6 Disinfection Alternatives PV/Normalized Benefit Ratio

CRITERIA	ALTERNATIVE 1 – SODIUM HYPOCHLORITE DISINFECTION	ALTERNATIVE 2 UV DISINFECTION	ALTERNATIVE 3 UV & SODIUM HYPOCHLORITE DISINFECTION	ALTERNATIVE 4 MULTI-BARRIER DISINFECTION
Total Weighted Score	34	40.5	34	34.5
Normalized Benefit Score	0.84	1.00	0.84	0.85
20-year PV Cost	$15,001,000	$7,122,000	$8,773,000	$12,218,000
PV / Normalized Benefit Ratio	$17,867,647	$7,121,000	$10,449,000	$14,342,870

4.0 Summary of Findings and Recommendations

Alternative 2 received the highest grade of the alternatives in the TBL analysis and has the lowest associated PV Cost. For these reasons, it is recommended that JCW plan to construct a 126 mgd UV facility at the time of expansion. The biggest penalty that Alternative 2 received during TBL analysis was in the Regulatory Risk category. Though the timeline is uncertain, as described in Section 1.3, there is a possibility of a future virus standard in NPDES permitting. If this standard were to develop between the time of this study and detailed design, the evaluation should be reconsidered when the specific indicator virus is determined. If the indicator virus is less responsive to UV radiation, Alternative 2 could be converted to a multi-barrier system similar to Alternative 4 by dosing final clarifier effluent with sodium hypochlorite.

Alternatives 1, 3, and 4 scored lower in the evaluation due to the continuous or intermittent use of chemicals associated with each. Discussions with JCW regarding the preference to move away from chemicals for safety reasons, along with the relatively high volume of truck traffic in chemical deliveries, and put these alternatives at a disadvantage in the TBL evaluation. O&M costs for a chemical feed operation also put these alternatives at a disadvantage to Alternative 2. The main contributor to O&M costs for Alternative 2 is power. A rate of $0.073/kW (the average rate at MCR WWTP from 2016-2019) is very low compared to other regions in the United States. Lastly, Alternatives 1, 3, and 4 carry the high capital cost associated with a CCB.

Based on conclusions described in this TM, UV disinfection will be carried forward as the disinfection treatment technology to be implemented at MCR WWTP at the time of expansion. This recommendation will be the assumption in future TMs. The design criteria for the UV Building will be as shown in Table 2-3. The site location and elevation of the UV Building will be determined in TM 8 – Site Optimization & MOPO.

DRAFT

MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 6

Biosolids Treatment

JCW NO. MCR1-BV-17-12
B&V PROJECT 403165

PREPARED FOR

JOHNSON COUNTY KANSAS

Wastewater

OCTOBER 7, 2020

BLACK & VEATCH

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Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
BTU	British Thermal Unit
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CANDO	Coupled Aerobic-anoxic Nitrous Decomposition Operation
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEA	Cost Effective Analyses
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
CIPP	Cured-in-place Pipe

Abbreviation	Meaning
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSBR	Continuous Sequencing Batch Reactor
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
D	
d	Day
DAF	Dissolved Air Flotation
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry-weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered

Abbreviation	Meaning
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand
ffCOD	Flocculated Filtered Chemical Oxygen Demand
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
ft	Feet
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
GGE	Gallons of Gas Equivalent
gpcd	Gallons per capita per day
gpd	Gallons per Day
gph	Gallons per Hour
gpm	Gallons per Minute
H	
HB	Hallbrook Facility
HDD	Horizontal Directional Drilling
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration
IC	Internal Combustion

Abbreviation	Meaning
IFAS	Integrated Fixed-Film Activated Sludge
in	Inches
IND	Industrial
INF	Influent
IP	Intellectual Property
IPS	Influent Pump Station
IR	Irrigation Use
IRR	Irrigation
IW	Industrial Water Supply Use
J	
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDHE	Kansas Department of Health and Environment
K_e	Light Extinction Coefficient
kWh	Kilowatt-hour
L	
L	Length, Liter
lb	Pound
LF	Linear Feet
LOMR	Letter of Map Revision
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter
mgd	Million Gallons per Day

Abbreviation	Meaning
min	Minute, minimum
mJ	Millijoules
MLE	Modified Ludzack Ettinger
MLSS	Mixed Liquor Suspended Solids
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow
mmBtu	Million British Thermal Units
mpg	Miles per Gallon
MPN	Most Probable Number
N	
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NFIP	National Flood Insurance Program
NH3-N	Total Ammonia
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
PV	Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
P	Phosphorous
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow
PE	Primary Effluent
PFE	Primary Filtered Effluent
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow

Abbreviation	Meaning
PIF	Peak Instantaneous Flow
PLC	Programmable Logic Controller
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day
pph	Pounds per Hour
PPI	Producer Price Index
ppw	Pounds per Week
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet-weather Flow
Q	
Q	Flow
R	
RAS	Return Activated Sludge
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
RPM	Revolutions per Minute
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SAF	Suspended Air Flotation
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
Sec	Second, Secondary
SF	Square Foot
SG	Specific Gravity
SLR	Solids Loading Rate
SMP	Stormwater Management Program, Shawnee Mission Park Pump Station
SND	Simultaneous Nitrification/ Denitrification

Abbreviation	Meaning
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station
SRT	Sludge Retention Time
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
TYP	Typical
U	
µg/L	micrograms per Liter
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet

Abbreviation	Meaning
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VFA	Volatile Fatty Acids
VFAs	Volatile Fatty Acids (Speciated)
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation – Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WL	Water Level
WK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

The purpose of this technical memorandum (TM) is to summarize the conceptual design of the biosolids treatment processes at Mill Creek Regional (MCR) wastewater treatment plant (WWTP). This TM includes a discussion of available thickening, digestion, dewatering, and biogas reuse technologies, dewatering alternatives analysis, design criteria of the selected technologies, footprint and layouts, and capital and operational and maintenance (O&M) costs.

For the dewatering alternatives evaluation, a life-cycle cost analysis was developed. The conceptual cost opinion was developed as a 20-year present value (PV) which includes the effects of inflation, time-value of money, and equipment O&M. A triple bottom line (TBL) analysis was then completed as the basis for selection of the biosolids treatment alternatives for further consideration. Social, environmental, and operational criteria were weighted and scored to determine the benefit-cost of each alternative.

This TM is one in a series of technical memoranda for the MCR Facility Plan. Additional treatment processes and site optimization of these treatment facilities will be outlined in future TMs.

1.1 BACKGROUND

Prior to this Facility Plan for MCR, an extensive alternative analysis was done for the Tomahawk Creek (THC) WWTP Expansion. The results of this analysis can be used to inform the planning of the MCR Expansion. THC WWTP is a good comparison because it is a similarly sized facility (19 million gallons per day (MGD) annual average (AA) flow), with similar wastewater characteristics, is owned and operated by JCW, and has actual market costs for treatment technologies provided by a Contractor.

In August of 2014 Johnson County Wastewater (JCW) retained Black & Veatch (B&V) for the project definition phase of the THC WWTP Expansion. The primary objective of the project definition phase was to confirm through alternative development and evaluation the optimal, proven treatment strategies throughout the WWTP for nutrient removal to meet current and anticipated future NPDES limits for design flows. Evaluation of these alternatives consisted of utilizing the JCW's TBL approach to evaluate non-economic factors in addition to developing capital and operating costs for each alternative. Each treatment process evaluation was presented to the JCW to select a recommended technology to be carried forward through design and construction.

After the project definition phase, the THC WWTP Expansion was continued into detailed design followed by construction. The construction is scheduled to be completed in 2021. During the detailed design phase some of the selected treatment technologies were re-evaluated and eventually revised as part of a value engineering effort. The treatment technologies that were part of the final design and eventually carried into construction serve as a valuable comparison for the MCR WWTP.

TM 1 established the design flows and loadings for the MCR WWTP. These design loadings were modeled to develop design biosolids production as shown in Table 1-1 below. The design loadings were used so size process equipment, tanks, and associated pumps. In general, pumps or groups of pumps are sized for the maximum month flow condition with the assumption that adjustable frequency drives will be used for turndown to meet process flow requirements.

Table 1-1 MCR Design Biosolids Production

PARAMETER	START-UP	ANNUAL AVERAGE	MAXIMUM MONTH
Primary Sludge			
Flow, mgd	0.18	0.59	0.77
Solids Load, ppd	7,710	24,520	31,920
% Solids	0.5%	0.5%	0.5%
WAS[1]			
Flow, mgd	0.42	0.37	0.56
Solids Load, ppd	8,170	16,030	27,210
% Solids	0.2%	0.5%	0.6%

[1] Assumes 30% surface WAS (from BNR) and 70 % Return Activated Sludge WAS

1.2 SUMMARY OF AVAILABLE TECHNOLOGIES AND SELECTION

1.2.1 Existing MCR Solids Treatment

Waste Activated Sludge (WAS) from the existing final clarifiers at the MCR WWTP is currently pumped to the end of Completely Mixed Cells 1 and 2. The sludge settles to the bottom of the partially mixed lagoons and dilute influent is circulated across the lagoon to reduce odor generation. The sludge is periodically dredged and land applied. This approach is not considered feasible for future development of the MCR WWTP due to the potential for odors, increased sludge production as flows and loadings increase, and removal of lagoon footprint to make room for the new treatment processes.

1.2.2 Primary Sludge Thickening and Fermentation

The selected mode of primary treatment for the future MCR WWTP project is primary clarification as described in TM 2 – Preliminary and Primary Treatment. Thickening and fermentation of primary sludge from the primary clarifiers provides a source of volatile fatty acids (VFAs) which are necessary for biological phosphorus removal in the secondary treatment process. Primary sludge fermentation provides a larger source of VFAs compared to other technologies such as mixed liquor suspended solids (MLSS) fermentation. Further, VFAs produced in primary sludge fermentation may be used to supplement the WASSTRIP process. WASSTRIP supports the release of phosphorus from polyphosphate accumulating organisms (PAOs) to the liquid phase, which is then directed to the phosphorus recovery system (see Section 2.3 for a more detailed description of WASSTRIP). WASSTRIP, an anaerobic process, is expected to produce some VFA, but addition of a supplemental source would increase process efficiency. Operationally, it is easier to supplement VFAs to the WASSTRIP process from primary sludge fermentation than from MLSS fermentation. The thickened primary sludge can then be sent to anaerobic digestion for additional solids destruction and biogas production. Three alternatives for primary sludge thickening and fermentation were evaluated for the THC WWTP, including centrifuge thickening and fermentation, separate gravity thickening and fermentation, and combined gravity thickening and fermentation.

Centrifuge thickening and separate fermentation was evaluated for the THC WWTP due to the reduced odor potential from completely enclosed centrifuges. However, centrifuges were determined to have a higher O&M cost than other alternatives due to significantly higher electricity use and maintenance requirements.

Separate gravity thickening and fermentation was evaluated due to the lower O&M cost and simpler operation of gravity thickeners. However, this alternative required the largest footprint and still had higher capital and O&M costs compared to combined gravity thickening and fermentation.

Combined gravity thickening and fermentation was also evaluated and ultimately selected due to the reduced footprint compared to separate thickening and fermentation, and reduced capital and O&M costs compared to both alternatives. Therefore, the conceptual design of the MCR WWTP primary sludge thickening and fermentation will be based on combined gravity thickeners and fermenters.

1.2.3 WAS Thickening

Similar to primary sludge, the sludge produced in the final clarifiers described in TM 3 – Secondary and Sidestream Treatment must also be treated. Centrifuge co-thickening was evaluated for the THC WWTP due to the reduced odor potential from completely enclosed centrifuges and the small facility footprint. Similar to primary sludge thickening however, centrifuges were determined to have a higher O&M cost than other alternatives due to significantly higher electricity use and maintenance requirements.

Rotary drum thickeners (RDTs) were evaluated and ultimately selected for the THC WWTP due to lower O&M cost and simpler operation. However, RDTs still require constant operator supervision. JCW staff indicated a preference for alternatives that did not require constant operator supervision to help reduce staffing requirements in 2nd and 3rd shifts.

The dissolved air floatation (DAF) process was identified as a thickening technology that could be operated without constant operator supervision. JCW currently operates a DAF system at their New Century Air Center and Blue River WWTPs. For planning purposes, the DAF process provides a conservative cost and footprint. Therefore, the conceptual design of the WAS thickening processes will be based on the DAF process. During preliminary design, emerging technologies similar to DAF, such as suspended air flotation (SAF), could also be evaluated.

1.2.4 Digestion

The THC WWTP utilized the existing mesophilic anaerobic digestion (MAD) process through the refurbishing of the existing digester tankage and control building. The conceptual design of the MCR WWTP digestion process will also be based on MAD with all new digestion facilities as the treatment plant currently doesn't have digestion facilities.

1.2.5 Dewatering

Centrifuge dewatering was selected for the THC WWTP due to its high capacity and small footprint. The THC WWTP dewatering centrifuges were sized to operate 24 hours per day, 5 days per week. The conceptual design of the MCR WWTP digested sludge dewatering will be based on operating 7 hours per day, 5 days per week to eliminate concerns with the 2nd and 3rd shifts similar to thickening. Two alternatives will be evaluated for the dewatering process at MCR WWTP, including centrifuges and belt filter presses. The evaluation includes both life cycle costs and social and environmental factors in a TBL analysis.

1.2.6 Phosphorus Recovery

Phosphorus recovery through struvite harvesting was evaluated for the THC WWTP but was not included in the final design and construction due to overall project cost constraints. For the purposes of the MCR WWTP Facility Plan, phosphorus recovery through struvite harvesting is included to develop a budgetary cost and footprint. The conceptual design of the phosphorus recovery process will be based on the Ostara Fx process. The Ostara Fx process includes a WASSTRIP reactor to release phosphorus from WAS and the Ostara Fx reactor to precipitate struvite from the phosphorus-rich sidestream.

1.2.7 Digester Gas Utilization

Several alternatives for digester gas utilization were evaluated for the THC WWTP, including Combined Heat and Power (CHP), Compressed Natural Gas (CNG), and digester heating. CHP was determined not to be feasible due to the long payback period (> 20 years) for the capital cost. Digester heating was ultimately selected for the THC WWTP due to existing facility and total project cost constraints. However, it was found that CNG for vehicle fuel would have a short payback period (<10 years) with the addition of Renewable Identification Number (RIN) credits. The conceptual design for the MCR WWTP will include CNG production for vehicle fuel for cost and footprint considerations as this appears to be the most cost-effective of the THC WWTP alternatives when constructing new digesters. There is a large diameter natural gas pipeline adjacent to the Mill Creek site. Pipeline injection of the natural gas could also be considered during preliminary design if the pipeline owner will accept the gas generated by the Mill Creek WWTP. Pipeline injection may require additional gas treatment to meet the utility's quality requirements, as well as additional costs associated with the pipeline connection, metering, and quality monitoring.

1.2.8 Biosolids Treatment Process

The overall biosolids treatment process was developed based on the individual technologies discussed above. The overall biosolids treatment process flow diagram for the MCR WWTP is shown in Figure 1-1 below. Overall, phosphorus is eliminated from the digestion process through WASSTRIP, which aids in dewaterability of the digested sludge, provides higher phosphorus recovery and reduces the potential for struvite formation in the digestion and dewatering process.

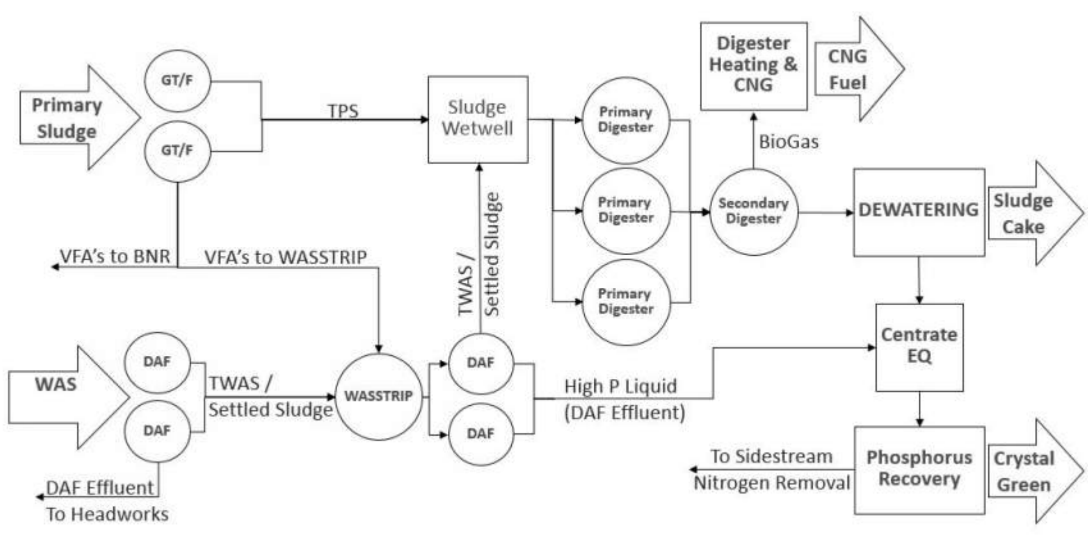


Figure 1-1 Biosolids Treatment Process Flow Diagram

2.0 Basis of Design Criteria

2.1 PRIMARY SLUDGE THICKENING/FERMENTATION

2.1.1 Gravity Thickener/Fermenter Design Criteria

The primary sludge thickener/fermenter process will consist of two circular tanks, each sized for 75% of the required volume in order to provide process and O&M flexibility, as opposed to the single gravity thickener sized for 100% of the required volume that was implemented at the THC WWTP. Each tank will be constructed of cast-in-place concrete and will include a flat aluminum walkable cover. A center column clarifier mechanism with sludge rake arms will collect sludge in a central hopper. Supernatant will be collected from the perimeter weir and will flow into an adjacent supernatant wetwell. The gravity thickener/fermenter will also have a full radius scum baffle with a scum beach. Scum will be collected from the water surface with a full radius scum beach and will flow into an adjacent scum wetwell.

Table 2-1 Primary Sludge Thickener/Fermenter Design Criteria

CRITERIA	VALUE
Number of Thickeners	2
Diameter, ft	55
Floor Slope	3:12
Surface Area, ft^2 (Each Tank)	3,600
Sidewater Depth, ft	17
Installed Horsepower, hp	2
Maximum Month influent flow rate, mgd (Each Tank)	0.385
Feed Solids, %	0.5
SRT, days	4
Effluent Total Solids, %	3

2.1.2 Thickener/Fermenter Pumping Design Criteria

Pumps and electrical equipment associated with the primary sludge thickener/fermenters will be located in the Thickening Building. The primary sludge gravity thickener/fermenters will have thickened sludge pumps and supernatant pumps. Thickened sludge pumps will be progressing cavity type. There will be a dedicated thickened sludge pump and sludge recycle pump for each gravity thickener/fermenter tank, the recycle pump is meant to reduce sludge stratification and enhance settleability. In addition, there will also be one common scum pump, and one common standby pump. All pumps will be the same for redundancy and ease of maintenance.

Table 2-2 Thickener/Fermenter Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	6 (2 Sludge, 2 Recycle, 1 Scum, 1 Standby)
Type	Progressing Cavity
Capacity, gpm (Each)	40
Motor rating, hp	7.5

Supernatant pumps will be horizontal end suction centrifugal type equipped with adjustable frequency drives. The pumps will draw suction from a common supernatant wetwell between the gravity thickener/fermenter tanks. Two duty pumps will pump supernatant to the BNR process and one duty pump will pump supernatant to the WASSTRIP Tank. A common standby pump will also be included.

Table 2-3 Supernatant Pumping Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	4 (3 duty, 1 standby)
Type	Non-clog, end suction centrifugal
Capacity, gpm (Each)	230
Motor rating, hp	15

2.1.3 Primary Treatment Technology Effects

In TM 2 - Preliminary and Primary Treatment, an alternative was evaluated for using cloth disk filters for primary treatment. This alternative was ultimately not selected because the use of cloth filters for primary treatment is considered an emerging technology at the time of this report. However, TM 2 also notes that the technology may continue to develop between the time of this report and the preliminary design phase for MCR potentially making it more favorable. The use of cloth disk filters would have a significant impact on the primary sludge thickening and fermentation process. Cloth disk filters have a higher solids capture, but also have a much higher backwash volume. Therefore, two stages of gravity thickeners would be required to thicken the primary sludge to the desired concentration for fermentation (i.e. the first stage would thicken from approximately 0.1% to 0.5% and the second stage would thicken from approximately 0.5% to 3%. Due to the higher solids capture, there would be more primary sludge solids produced, but less WAS produced. Therefore, if disk filters are ultimately selected during preliminary design, the biosolids treatment process should be re-evaluated.

2.2 WAS THICKENING

WAS from the secondary treatment process will be thickened using the DAF process. The DAF Process was selected for planning purposes because JCW is familiar with the technology and is comfortable operating it with minimal operator supervision.

WAS will be thickened prior to the WASSTRIP tank to increase the phosphorus and nitrogen concentrations in the sidestream flow, which reduces the hydraulic load and can improve the efficiency of the sidestream treatment processes.

The first stage DAF thickening will consist of two circular DAF tanks, each sized for 50% of the design solids loading rate without polymer addition. Each DAF tank will include a domed cover to contain odors and allow operators to enter the DAF for visual inspection of the process. The DAF tanks will have a common sludge wetwell and a common effluent wetwell. A portion of the effluent will be recycled to the saturation tanks to provide the dissolved air feed. The DAF has been sized to operate without polymer addition, however space has been allocated in the preliminary Thickening Building layout for polymer feed equipment to be installed in the future, if needed. Polymer addition would allow one DAF to operate at a higher solids loading rate when the other DAF is out of service. The first stage DAF design criteria are provided in Table 2-4 below.

2.2.1 First Stage DAF Design Criteria

Table 2-4 First Stage DAF Design Criteria

PARAMETER	DESIGN CRITERIA
DAF Tanks	
Number of Units	2
Diameter, ft	45
Sidewater Depth, ft	13
WAS Flow, gpm (Each)	255
Pressurized Recycle Stream, %	20
DAF Recirculation Pumps	
Type	Non-clog, end suction centrifugal
Capacity, gpm (Each)	60
Motor rating, hp	10

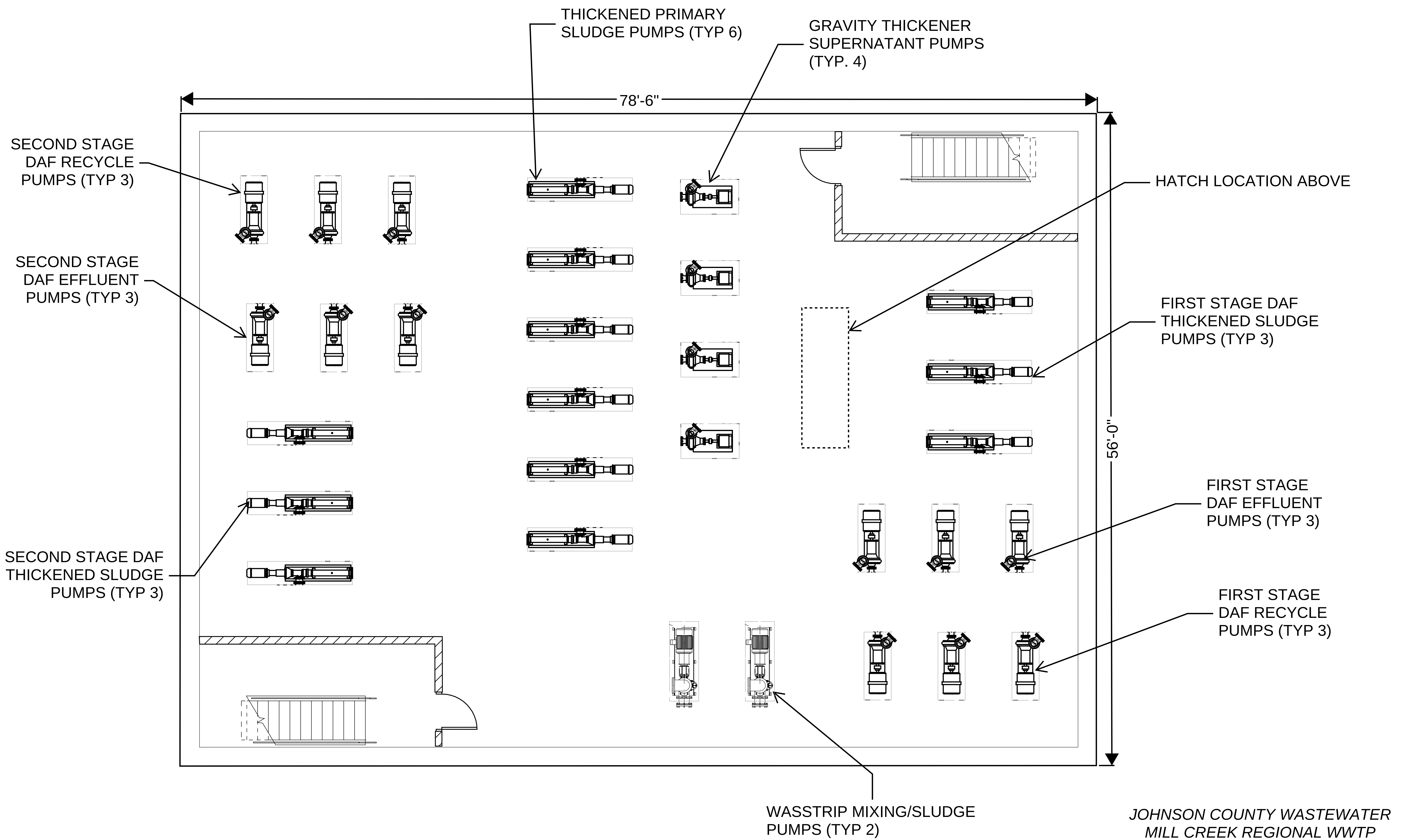
Thickened and settled sludge from the First Stage DAF will be pumped to the WASSTRIP tank by progressing cavity sludge pumps. Two duty pumps will be provided for flexibility and turndown with a single standby pump. The DAF Effluent or underflow, that is separated from the WAS will be pumped to the Headworks building by centrifugal pumps. Two duty and one standby pump will be provided for the underflow pumps. The First Stage DAF Pumps will be located in the basement of the Thickening Building as shown in Figure 2-1 and Figure 2-2. The First Stage DAF Sludge Pump and Effluent Pump design criteria are provided in Table 2-5 and Table 2-6 below.

Table 2-5 First Stage DAF Sludge Pumping Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	3 (2 duty, 1 standby)
Type	Progressing Cavity
Capacity, gpm (Each)	60
Motor rating, hp	7.5

Table 2-6 First Stage DAF Underflow Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	3 (2 duty, 1 standby)
Type	Non-clog, end suction centrifugal
Capacity, gpm (Each)	285
Motor rating, hp	7.5



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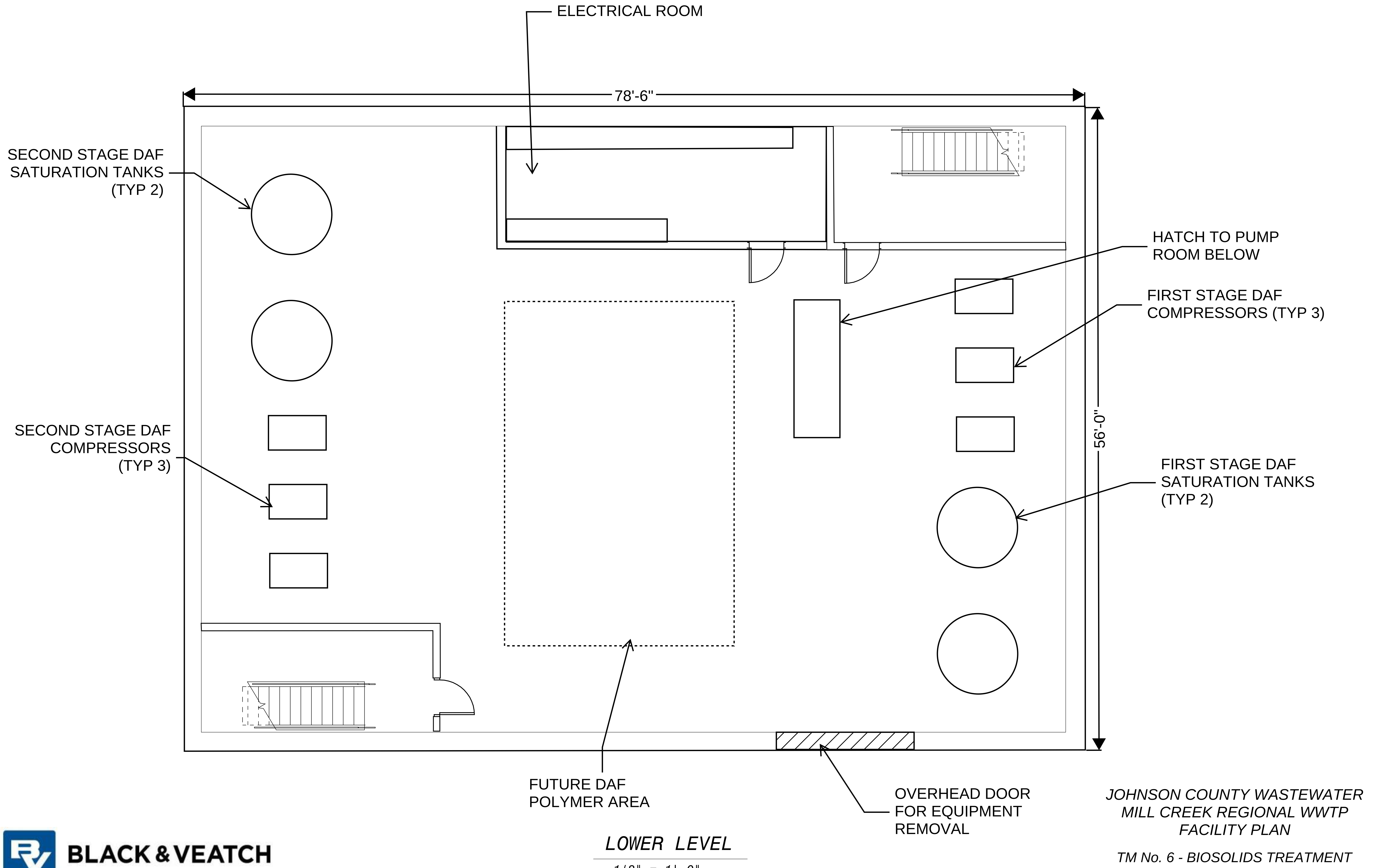
JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 6 - BIOSOLIDS TREATMENT
THICKENING BUILDING LOWER LEVEL PLAN
FIGURE 2-1



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JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 6 - BIOSOLIDS TREATMENT
THICKENING BUILDING GROUND LEVEL PLAN
FIGURE 2-2

2.3 PHOSPHORUS STRIPPING

The WASSTRIP process releases luxury phosphorus from organisms in the WAS so that it can be chemically precipitated as struvite in the Ostara process. Thickening the WAS prior to the WASSTRIP process decreases the volume required in the WASSTRIP tank to achieve the desired hydraulic retention time (HRT) and increases the sludge concentration, providing better P release. Primary Sludge fermentate from the gravity thickener/fermenter can be added to the WAS to provide VFAs and facilitate quicker phosphorus release. A secondary thickening/separation process follows the WASSTRIP process in order to separate the high-phosphorus liquid stream from the solids stream. The WASSTRIP process and the secondary thickening process provide a consistent source of phosphorus to the Ostara process to improve phosphorus recovery and process reliability. Operating the Ostara process solely on digested sludge centrate, without the WASSTRIP and secondary thickening processes would reduce overall phosphorus removal/recovery and could have a negative impact on process reliability.

2.3.1 WASSTRIP Design Criteria

The WASSTRIP tank is an anaerobic, mixed tank. The tank will be circular, cast-in-place concrete construction with a flat walkable cover for access and to contain potential odors. The tank will be mixed by a submersible mixer. The WASSTRIP tank design criteria are provided in Table 2-7 below. The total tank volume was rounded up for conceptual design. The WASSTRIP process could also accept approximately half of the primary sludge fermentate flow. However, under normal operating conditions this will likely be significantly less than the maximum.

Table 2-7 WASSTRIP Tank Design Criteria

DESIGN PARAMETER	VALUE
Thickened WAS flow rate, mgd	0.160
Thickened WAS thickness, % solids	2
Minimum hydraulic retention time (HRT), hours	12
WASSTRIP volume, gal	90,000
WASSTRIP Tank Diameter, ft	30
WASSTRIP Tank Depth, ft	17
Maximum Primary Sludge Fermentate flow rate, mgd	0.33
Mixer	
Quantity	1
Mixer Type	Submersible
Mixer Motor, hp	2.5

2.3.2 WASSTRIP Pumping Design Criteria

Sludge from the WASSTRIP process will be pumped to the Second Stage DAF to separate the solids from the phosphorus-rich liquid stream. The pumps will be located in the basement of the

Thickening Building as shown in Figure 2-1. The WASSTRIP pumping design criteria are provided in Table 2-8 below.

Table 2-8 WASSTRIP Pumping Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	2 (1 sludge transfer, 1 standby)
Type	Progressing Cavity
Capacity, gpm (Each)	170
Motor rating, hp	10

2.3.3 Second Stage DAF Design Criteria

The Second Stage DAFs will separate the WAS Solids from the phosphorus-rich liquid stream from the WASSTRIP tank. The TWAS and settled solids will be pumped to the sludge wetwell for digestion, and the high phosphorus liquid stream (DAF effluent/underflow) will be pumped to the Centrate EQ tank for phosphorus recovery. The Second Stage DAFs will be the same size as the first stage DAFs due to the similar solids loading rates. The Second Stage DAF pumps will also have a similar arrangement as the first stage DAF pumps and will be located in the basement of the Thickening Building as shown in Figure 2-1. The Second Stage DAF and associated pumping design criteria are provided in Table 2-9, Table 2-10, and Table 2-11.

Table 2-9 Second Stage DAF Design Criteria

PARAMETER	DESIGN CRITERIA
DAF Tanks	
Number of Units	2
Diameter, ft	45
Sidewater Depth, ft	13
Maximum WASSTRIP Flow, gpm (Each unit)	170
Pressurized Recycle Stream, %	20
DAF Recirculation Pumps	
Number of Units	3 (2 duty, 1 standby)
Type	Non-clog, end suction centrifugal
Capacity, gpm (Each)	15
Motor rating, hp	10

Table 2-10 Second Stage DAF Sludge Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	3 (2 duty, 1 standby)
Type	Progressing Cavity
Capacity, gpm (Each)	60
Motor rating, hp	7.5

Table 2-11 Second Stage DAF Underflow Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	3 (2 duty, 1 standby)
Type	Non-clog, end suction centrifugal
Capacity, gpm (Each)	40
Motor rating, hp	10

2.4 DIGESTION

Mesophilic anaerobic digestion (MAD) will be used to stabilize both primary sludge and TWAS. The MAD digesters are designed to meet Class B biosolids requirements for pathogen reduction. The digested solids will be suitable for landfill or bulk land application.

The thickened primary sludge and post-WASSTRIP sludge will be combined in a sludge blend tank prior to digestion. The sludge blend tank will be a below-grade cast-in-place concrete basin adjacent to the digester control building in order to minimize the length of suction piping for the digester feed pumps. The volume of the sludge blend tank will be minimized to one-hour retention time to allow homogeneous mixing while reducing odor potential. A submersible mixer will be installed in the sludge blend tank to thoroughly mix the two sludges and provide a more consistent sludge to feed the digesters. A ferric chloride feed will be provided to the sludge blend tank that can be used to reduce hydrogen sulfide formation in the digesters. The design criteria of the sludge blend tank are provided in Table 2-12 below.

Table 2-12 Sludge Blend Tank Design Criteria

PARAMETER	DESIGN CRITERIA
Thickened Primary Sludge Flow, mgd	0.103
Thickened WAS Flow, mgd	0.103
HRT, hr	1
Volume, cf	1,150
Mixer	
Quantity	1

Mixer Type	Fixed, top entry
Mixer Motor, hp	15

2.4.1 Digester Design Criteria

Four digester tanks will be provided. Digesters 1-3 will serve as primary digester tanks and Digester 4 will serve as a secondary digester, providing digested sludge storage. The Digesters will be circular concrete construction. Wire-wrapped precast concrete construction has been assumed for construction of the Digesters. Fixed steel covers will be provided for Digester 1-3 to contain odors and collect biogas. A gas holding membrane cover will be provided for Digester 4.

Table 2-13 Digester Design Criteria

PARAMETER	DESIGN CRITERIA
No. Primary Digesters	3
No. Secondary Digesters	1
Diameter, ft	64
Depth, ft	45
Volume Each Digester, kcf	141

2.4.2 Digester Heating and Mixing Design Criteria

Digester heating will be provided for the primary digesters using combination heater/boiler units and sludge heat exchangers similar to what is used at the THC WWTP. Digester heating and mixing equipment will also be provided for Digester 4 so that it can be used as a primary digester in the event that one of the other digesters has to be taken out of service for maintenance or cleaning. The heating and mixing equipment for Digester 4 will also be piped to serve as standby equipment for the other digesters. Sludge circulation pumps will circulate sludge from the primary digesters through the heat exchangers to maintain mesophilic temperatures. The boilers, heat exchangers, and circulation pumps will be located in the Digester Control Building shown in Figures 2-3 and 2-4.

Table 2-14 Sludge Circulation Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Pumps	4 (3 duty, 1 standby)
Type	Non-Clog End-Suction Centrifugal
Capacity, gpm (Each)	200
Motor rating, hp	10

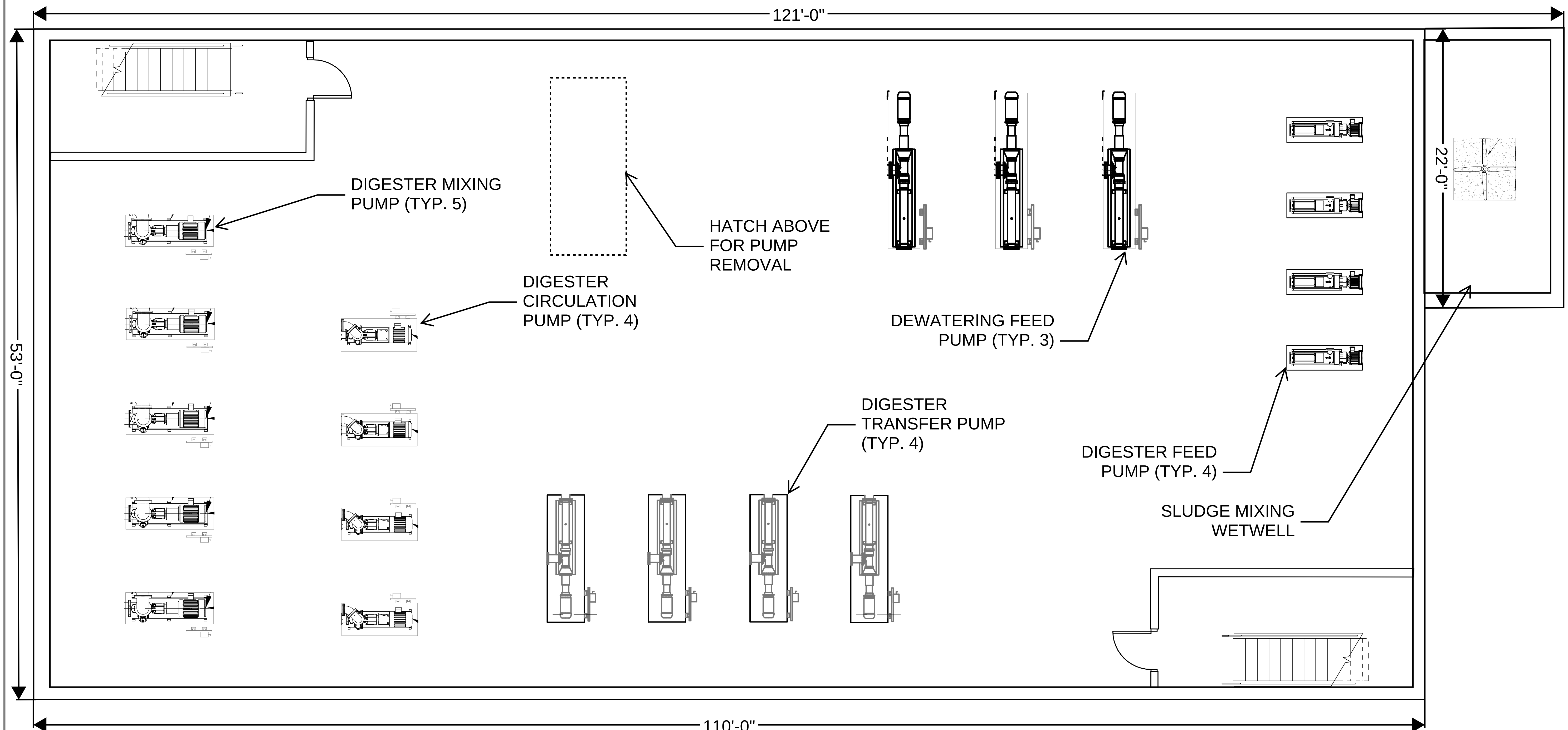
2.4.3 Digester Pumping Design Criteria

The digester feed pumps will pull mixed sludge from the sludge blend tank and pump it to the three primary digesters. The pumps will be progressing cavity type with adjustable frequency drives.

There will be three duty pumps, with each pump feeding one of the primary digesters. A common standby pump will also be provided. The digester feed pumps will be located in the Digester Control Building shown in Figure 2-3. The digester feed pump design criteria are provided in Table 2-15 below.

Table 2-15 Digester Feed Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Pumps	4 (3 duty, 1 standby)
Type	Progressing Cavity
Capacity, gpm (Each)	50
Motor rating, hp	10



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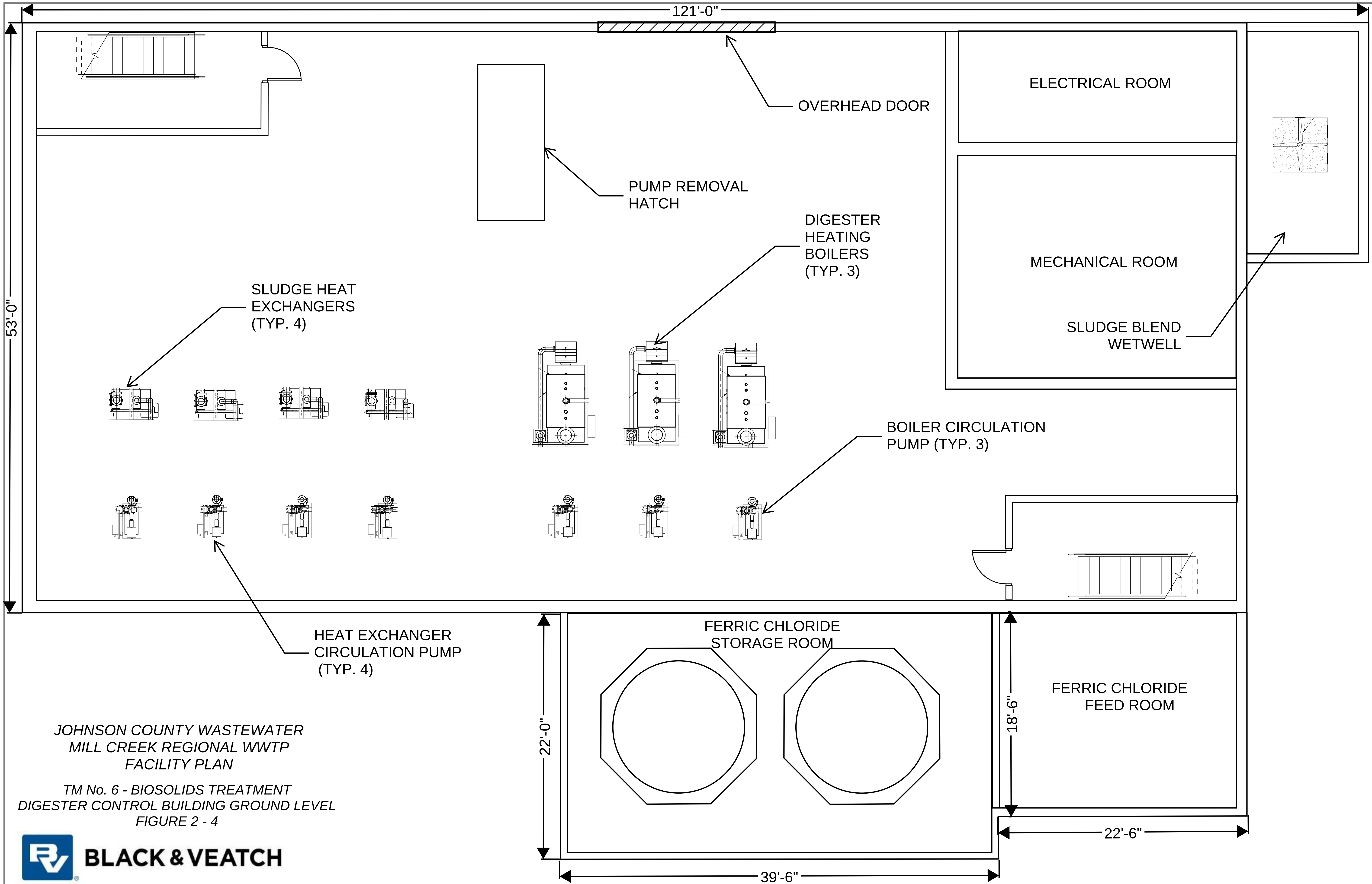
JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 6- BIOSOLIDS TREATMENT
DIGESTER CONTROL BUILDING LOWER LEVEL
FIGURE 2 - 3



JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 6 - BIOSOLIDS TREATMENT
DIGESTER CONTROL BUILDING GROUND LEVEL
FIGURE 2 - 4

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A pumped mixing system was assumed for the purposes of this report because it has a larger footprint leading to a more conservative capital cost when compared to other alternatives such as submersible mixers or vertical linear motion mixers. The digester mixing pumps will be chopper style pumps with fixed discharge nozzles within the digester tanks. The digester mixing pump design criteria are provided in Table 2-16 below.

Table 2-16 Digester Mixing Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Pumps	5 (4 duty, 1 standby)
Type	Chopper Pumps
Capacity, gpm (Each)	3,000
Motor rating, hp	40

The sludge transfer pumps will move digested sludge from Digesters 1-3 to Digester 4. Piping will be installed to transfer sludge from Digesters 1 and 2 to Digester 3 as well, so that Digester 3 can be used as a sludge storage tank in the event that Digester 4 is out of service. Design criteria for the sludge transfer pumps are provided in Table 2-17 below.

Table 2-17 Sludge Transfer Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Pumps	4 (3 duty, 1 standby)
Type	Progressing Cavity
Capacity, gpm (Each)	50
Motor rating, hp	10

The Dewatering Feed Pumps will draw digested sludge from Digester 4 and transfer it to the Dewatering Building. Piping will also be installed to allow sludge to be drawn from Digester 3 in the event that Digester 4 is out of service. Two in-line grinders will be installed on a common header upstream of the dewatering feed pumps to grind any debris from the digesters before it reaches the dewatering equipment. Two alternatives for dewatering equipment were evaluated, as discussed in Section 2.5. Two sets of dewatering feed pump design criteria were developed due to the significant differences in configuration and flow rates for the different dewatering equipment. The Centrifuge dewatering feed pump configuration was ultimately used for layout and cost purposes because the Centrifuge dewatering was the preferred dewatering alternative.

Three dewatering feed pumps will be provided to feed the three dewatering centrifuges. Each pump will be sized to match the centrifuge capacity, providing the same level of redundancy as the centrifuges. The centrifuge dewatering feed pump design criteria are provided in Table 2-18 below. The Dewatering Feed Pumps will be located in the Digester Control Building to minimize the length of suction piping from Digester 4.

Table 2-18 Centrifuge Dewatering Feed Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Pumps	3 (1 duty, 2 standby)
Type	Progressing Cavity
Capacity, gpm (Each)	450
Motor rating, hp	30

The belt filter press dewatering pump alternative includes eight dewatering feed pumps to feed the eight dewatering belt filter presses. Each pump is sized to match the hydraulic capacity of a single belt filter press. The belt filter press dewatering feed pump design criteria are provided in Table 2-19 below.

Table 2-19 Belt Filter Press Dewatering Feed Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Pumps	8 (6 duty, 2 standby)
Type	Progressing Cavity
Capacity, gpm (Each)	120
Motor rating, hp	7.5

2.5 DEWATERING

Digested sludge will be dewatered to produce a class B biosolids cake for land application. The centrate stream from the dewatering operation is high in phosphorus and nitrogen and will be combined with the WASSTRIP liquid stream for phosphorus recovery and sidestream nitrogen removal. Digested sludge dewatering is scheduled for single-shift, 5-day per week operation. Therefore, sludge will be equalized in Digester 4 and sent to dewatering at a higher rate during the week. The dewatering operation is set at 7 hours per day to allow time for startup and shutdown at the beginning and end of each shift. The dewatering feed criteria are shown in Table 2-20 below.

Table 2-20 Dewatering Feed Criteria

PARAMETER	ANNUAL AVERAGE	MAX MONTH
Digested Sludge Flow Rate, gpm	84	123
Digested Sludge Solids, %	2.2	2.2
Digested Sludge, ppd	22,205	32,540
Digested Sludge, ppw	155,435	227,775
Schedule Adjusted Loading		
Shifts per Week	5	5
Hours per Shift	7	7
Flow Rate, gpm	403	590
Solids Loading Rate, pph	4,440	6,510

Two alternatives are evaluated in this report for dewatering: centrifuges and belt filter presses. Each alternative is discussed in detail below.

2.5.1 Alternative 1 - Centrifuges

Centrifuges were evaluated as a dewatering alternative because they were the selected technology for dewatering at the THC WWTP. Centrifuges operate by rotating a cylindrical bowl at high RPM to create a centrifugal force that separates the solids and liquids from the digested sludge.

The centrifuges have been sized to provide all dewatering from a single duty unit. Due to the single shift operation at the MCR WWTP, a larger centrifuge will be required than what is utilized at the THC WWTP. The dewatering centrifuge design criteria are provided in Table 2-21 below. The solids loading rate and hydraulic loading rate shown are an average of several manufacturers. The hydraulic loading rate is the limiting factor for centrifuge sizing. During the projected maximum month conditions, the single duty centrifuge will need to be operated for approximately 9 hours per day, or one of the standby units utilized periodically to catch up with the digested sludge produced.

In order to perform significant maintenance or repair on centrifuges, JCW staff must remove them and ship them to the manufacturer's service representative. Therefore, a large bridge crane is needed in the centrifuge room as well as a hatch to the truck bay below to allow the centrifuges to be lowered onto a truck. Because of the time required to remove, service, and reinstall the centrifuges, two standby units will be provided so there is still full redundancy when one unit is removed for service. A layout of the centrifuge room is provided in Figure 2-5 below.

Centrifuges are completely enclosed, significantly reducing potential odors. Each centrifuge is directly ventilated while in operation and the airflow is sent to an odor control system. This significantly reduces the size of odor control system required compared to whole-room ventilation and odor control.

Table 2-21 Dewatering Centrifuge Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Duty Units	1[1]
Number of Standby Units	2
Bowl Diameter, in	29
Hydraulic Loading Capacity, gph per unit	27,000
Solids Loading Capacity, pph per unit	5,400
[1] One Centrifuge would need to be operated for 9 hrs/day, 5 days per week in order to meet Ultimate Maximum Month loading.	

2.5.2 Alternative 2 - Belt Filter Presses

Belt Filter Presses were evaluated as a dewatering alternative because JCW maintenance staff have experience and are comfortable disassembling and maintaining them in place. Belt filter presses operate by filtering solids through large cloth belts. The solids are retained on the belt while water escapes through the fabric and is collected in a trough below. Belt filter presses operate in two stages. In the gravity belt stage, sludge is deposited on an open belt and liquid falls through the belt by gravity. In the pressure stage, sludge is pressed between two belts that pass over several rollers, pressing additional water from the sludge to produce a sludge cake. A three-belt press was used for the conceptual design for this report. A three-belt press allows for separate control of the gravity stage and pressure stage.

Belt filter presses have a significantly lower solids loading rate compared to centrifuges. For the purpose of this evaluation a 2-meter wide belt was assumed to be the largest size that can be easily maintained by JCW staff. The dewatering belt filter press design criteria are provided in . Because of the lower solids loading rate the belt filter presses are solids limited instead of being hydraulically limited like the centrifuges. As such, 6 duty belt filter presses are required to meet the projected maximum month digested sludge production. For comparison purposes, 2 standby units have also been provided to provide the same level of redundancy in the event that one of the conveyors is removed from service, effectively removing two presses from service.

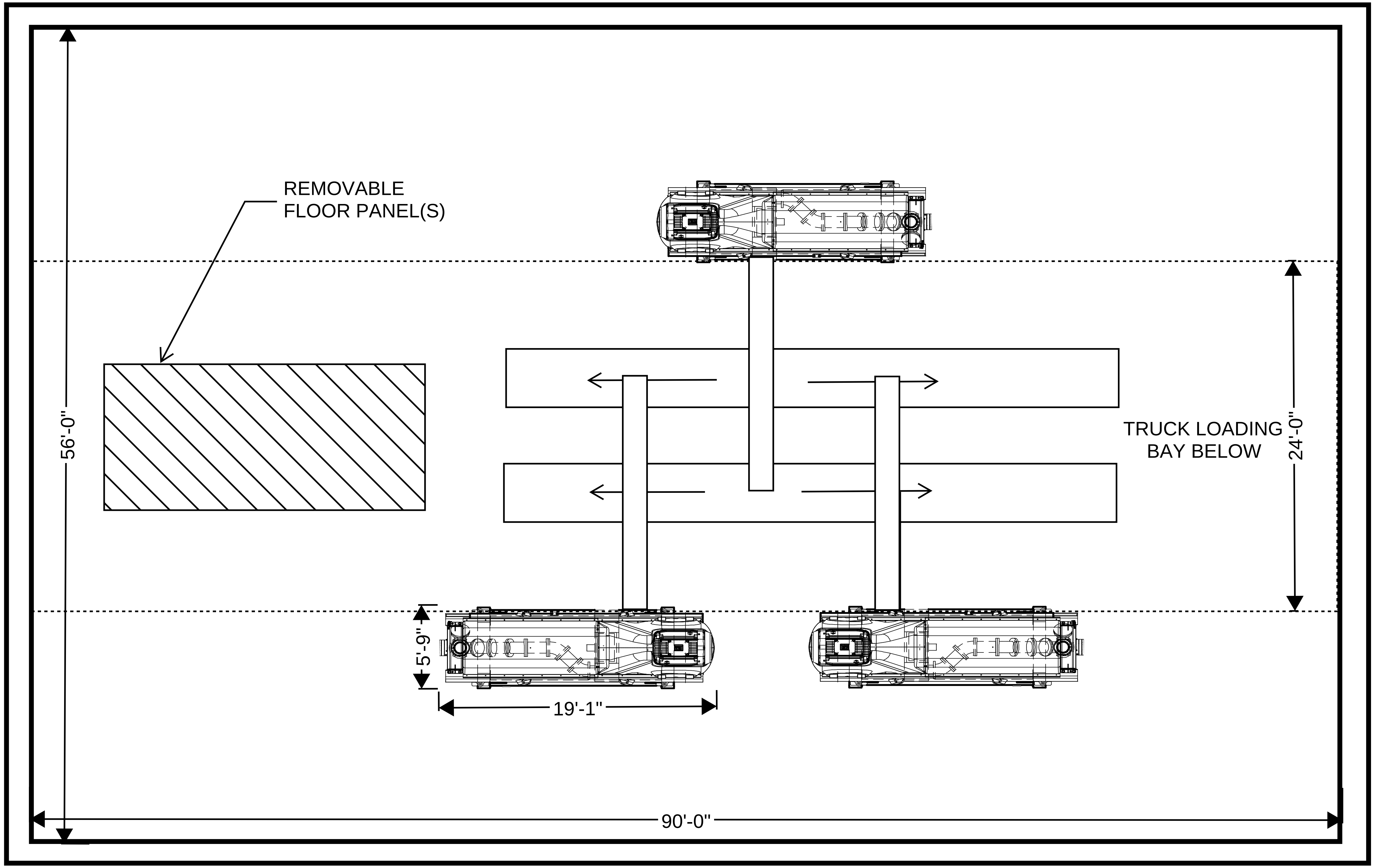
A layout of the Belt Filter Press room is provided in Figure 2-6. In order to service the belt filter presses, rollers must be removed from the side of the unit. Therefore, the units have been clustered in pairs, with a wide enough space between clusters to remove the rollers from one side of each press. There are several different configurations of belt filter press provided by different manufacturers. Some configurations have a raised gravity belt stage. Therefore, a raised platform is provided for each pair of presses to aid in observing and operating the presses. Because the belt filter presses can be maintained in place, equipment access is only required when the units are installed and when they are replaced at the end of their useful life. Therefore, large windows will be provided in the press room. The equipment would be installed through the window openings before the windows are installed. At the end of the equipment's useful service life, the windows would be temporarily removed to allow the equipment to be replaced. In order to provide the same level of redundancy in the dewatered solids conveyors, a conveyor is provided for every pair of belt filter presses. Therefore, if one conveyor is out of service, the other six belt filter presses will still be able to operate and maintain maximum month dewatering capacity.

Several belt filter press manufacturers have begun fabricating clear plexiglass enclosures that cover the portions of the machine with exposed sludge. The air within these enclosed spaces can then be vented to an odor control unit. This greatly reduces the airflow that must be treated by odor control, significantly reducing the capital and operating cost compared to providing high rate ventilation and odor control for the entire dewatering room. However, JCW staff have indicated that there are several operational issues with these enclosures, and they were not considered in this evaluation. As the technology continues to develop, advances in odor control for belt filter presses should be considered.

Belt filter presses are typically a significant source of odors because of their open design. In order to manage the odors, the entire dewatering room will be ventilated at a higher rate (12 air changes per hours compared to 6 air changes per hour) and exhaust air will be sent through an odor control system. The large footprint and high ventilation rate require a significantly larger odor control system compared to centrifuges. This increases the capital cost and operating cost of the odor control system for belt filter presses. Higher heating costs are also anticipated in the winter due to the higher ventilation rate, however this cost was not evaluated in this report.

Table 2-22 Dewatering Belt Filter Press Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Duty Units	6
Number of Standby Units	2
Belt Width, meters	2
Hydraulic Loading Capacity, gph per unit	7,200
Solids Loading Capacity, pph per unit	1,200



JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 6 - BIOSOLIDS TREATMENT
CENTRIFUGE DEWATERING LAYOUT
FIGURE 2 - 5

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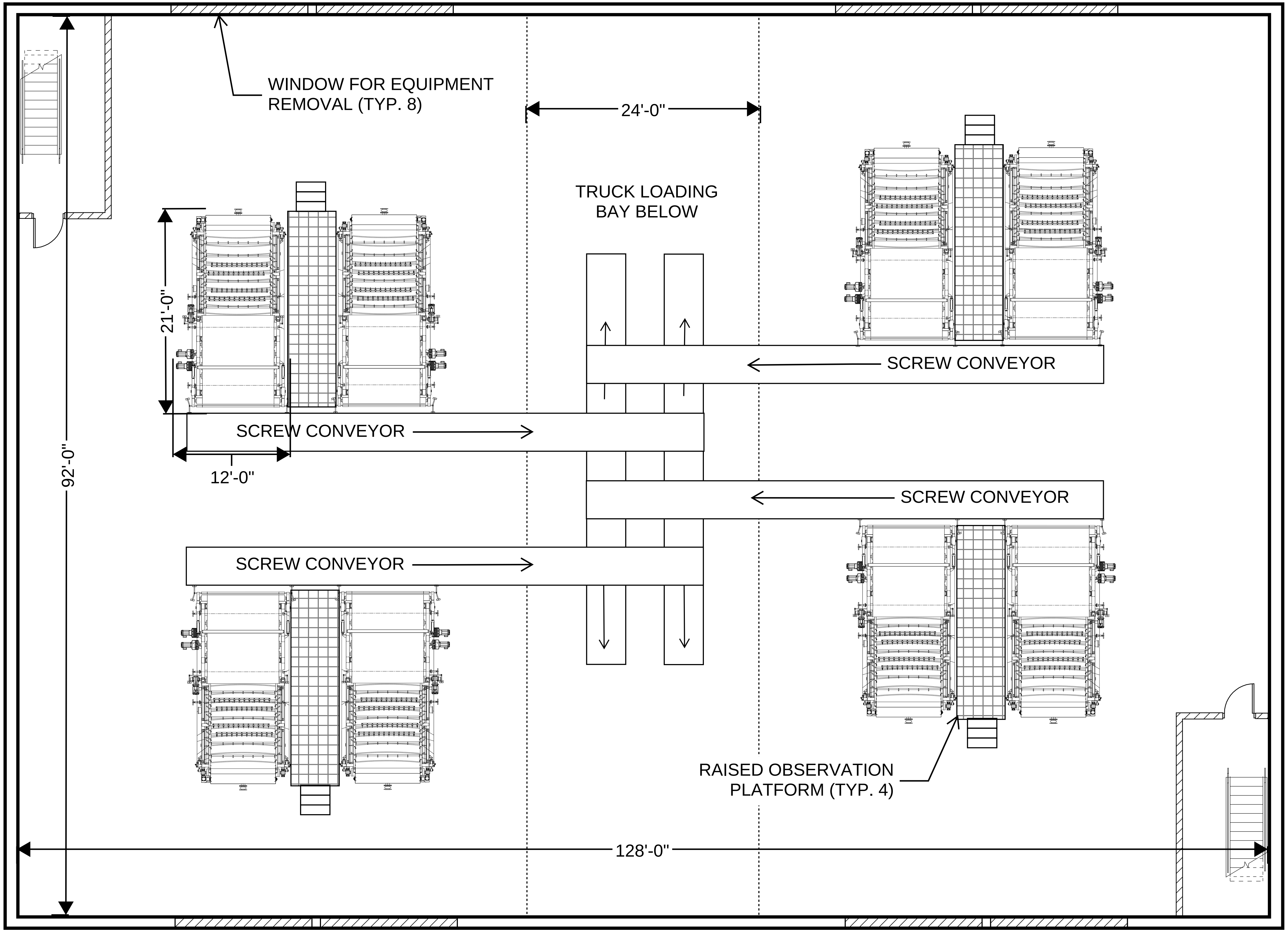

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MILL CREEK REGIONAL
WWTP FACILITY PLAN
TM No. 6 - BIOSOLIDS TREATMENT
BELT FITER PRESS DEWATERING LAYOUT
FIGURE 2 - 6

2.5.3 Cost Analysis

Preliminary capital and O&M Costs were developed for the two dewatering alternatives. The cost analysis evaluated the significant items that were different between the alternatives, including equipment costs, footprint of the dewatering room, and odor control. It was assumed that ancillary spaces and equipment would be similar enough between the alternatives that they would not create a significant difference in the costs.

The 20-year present value for each of the dewatering alternatives are summarized in Table 2-23. Present value estimates are based on the following additional assumptions:

- Cost year basis: 2020
- Nominal Discount Rate: 3.10%
- Inflation Rate: 1.90%
- Resulting Net Discount Rate: 1.20%

To calculate the total O&M cost over the 20-year life cycle, the annual O&M cost for each year is calculated by multiplying the previous year's annual O&M cost by the inflation rate. That annual O&M cost for that specific year is then corrected back to 2020 dollars, and the nominal discount rate is applied. The sum of all the annual present values is the overall present value O&M cost over 20-years.

Table 2-23 Dewatering Alternatives Costs

PARAMETER	ALTERNATIVE 1 – CENTRIFUGES	ALTERNATIVE 2 – BELT FILTER PRESSES
Capital Cost	$5,476,000	$11,350,000
Annual O&M Cost	$250,500	$359,000
O&M PV (20 years)	$4,673,000	$6,645,000
Total Life Cycle Cost	**$10,113,000**	**$17,995,000**

In summary, the capital cost associated with belt filter presses is higher due to the increased equipment cost and the cost of the larger building footprint. The O&M costs between the two alternatives are similar. Centrifuges have a much higher electrical cost and more polymer usage, but belt filter presses have a higher equipment maintenance cost as there are more presses, sludge feed pumps, and polymer feed pumps to maintain.

It is important to note that salvage value has not been included in this PV evaluation. Although there would be a salvage value associated with the structure in the 20-year PV life cycle for each of these alternatives, it is estimated that the similarity between structures would result in an across the board increase of similar magnitude for all alternatives. Since the goal of the total PV is to differentiate alternatives, salvage value has not been included.

2.5.4 Triple Bottom Line Analysis

The dewatering alternatives were evaluated through a TBL analysis. By factoring social and environmental considerations into the analysis along with economic information expressed as present value,(PV), a more thorough comparison of alternatives can be achieved. The benefit score was then combined with the PV to determine the benefit-cost of each alternative. The TBL criteria below in Table 2-24 were developed with JCW to capture MCR specific concerns and remain consistent with similar past evaluations.

Table 2-24 TBL Evaluation Criteria and Descriptions

CRITERIA	DESCRIPTION
Flexibility / Turndown	Is alternative flexible enough to successfully adjust to changing conditions (i.e. flow and load)? How much can be treated through the process?
Performance Reliability	Are there adjustable controls, process options, and/or equipment features available for operators to respond to an upset? Is alternative resistant to an upset, and what are the consequences if upset does occur? Is the alternative a proven technology?
Operational Complexity / Maintenance	How complex is the alternative to operate, control and maintain? Does the alternative rely on more system components operating together? Are there major scheduled replacements and cleanings?
Layout / Constructability	How easily and cost-effectively can the alternative be phased to meet the start-up and construction constraints? How well does the alternative fit on the site? Do the facilities lay out in an orderly fashion (e.g., do trucks have to drive to through several facilities in order to access their final destination)?
Social Impacts	How well does the alternative prevent off-site impacts to public perception such as truck traffic, noise, odor, visual aesthetics, etc. and can these impacts be easily mitigated? (Impacts from construction activities are excluded.)
Environmental Impacts	How well does the alternative minimize the impact to the environment in terms of carbon footprint (during construction and use phase), ecosystem quality, and resource use?
Safety	How well does the alternative minimize safety risks to the plant staff and the public and can the risks be mitigated?
Ease of Regulatory Acceptance	How difficult will it be to obtain EPA and KDHE regulatory acceptance of the alternative? Could alternative acceptance be achieved in desired schedule?

Table 2-25 provides a summary of the weighted scores for the dewatering alternatives. A ranking of 5 means this is the most important, or most positive impact. A ranking of 1 means this is the least important, or most negative impact.

Table 2-25 Dewatering Alternatives Triple Bottom Line Scoring

CRITERIA	RELATIVE WEIGHT	ALTERNATIVE 1 – CENTRIFUGES		ALTERNATIVE 2 – BELT FILTER PRESSES	
		Ranking	Weighted Score	Ranking	Weighted Score
Flexibility / Turndown	15%	3	4.5	4	6
Performance Reliability	20%	3	6	3	6
Operational Complexity / Maintenance	20%	2	4	4	8
Layout / Constructability	10%	4	4	3	3
Social Impacts	10%	3	3	3	3
Environmental Impacts	10%	3	3	3	3
Safety	10%	3	3	3	3
Ease of Regulatory Acceptance	5%	3	1.5	3	1.5
Total Weighted Score	100%	29		33.5	
Note: Rankings: 5 = Most Important or most positive impact. 1 = Least Important or most negative impact.					

Alternative 2 is rated higher in Flexibility/Turndown due to the operational flexibility to run fewer belt filter presses during periods of lower sludge production. Both alternatives are considered equal for performance reliability. While Alternative 1 has a much higher total capacity, the long timeframe of equipment being out of service for maintenance is considered to negate this advantage. Alternative 2 is rated higher for Operational Complexity/Maintenance because belt filter presses are simpler to operate and can be maintained by JCW staff, while centrifuges must be removed and sent away for major maintenance. Both alternatives are considered equal for Social Impacts as there is negligible difference between truck traffic and odor potential. Both alternatives are considered equal in terms of Safety as well. Alternative 1 requires the centrifuges to be lifted and removed periodically, which has significant safety concerns. However, the open design of belt filter presses and their moving belts leads to safety concerns for staff during normal operation. Both alternatives are considered equal for Ease of Regulatory Acceptance as they are both widely used technologies.

2.5.5 Cost/Benefit Scoring

The sum of the TBL scoring can be converted to the normalized benefit score based upon the highest scoring alternative. The benefit scores for each alternative are then divided into the respective PV to express the benefit score in economic terms. Table 2-26 contains the PV to the normalized benefit ratio for the dewatering alternatives.

Table 2-26 Dewatering Alternatives PV / Normalized Benefit Ratio

CRITERIA	ALTERNATIVE 1 – CENTRIFUGES	ALTERNATIVE 2 – BELT FILTER PRESSES
	Weighted Score	Weighted Score
Flexibility / Turndown	4.5	6
Performance Reliability	6	6
Operational Complexity / Maintenance	4	8
Layout / Constructability	4	3
Social Impacts	3	3
Environmental Impacts	3	3
Safety	3	3
Ease of Regulatory Acceptance	1.5	1.5
Total Weighted Score	29	33.5
Normalized Benefit Score	0.87	1
PV Cost	$ 10,113,000	$ 17,995,000
PV/ Normalized Benefit Ratio	**$ 11,624,000**	**$ 17,995,000**

2.5.6 Triple Bottom Line Summary and Future Considerations

Based on the TBL Analysis, Centrifuges are the preferred alternative. While Alternative 2 scored higher in TBL analysis, the difference was not enough to make up for the significantly higher costs. However, during preliminary design the importance of in-house maintenance, as well as the operational experience with the newer centrifuges recently installed at the THC WWTP should be evaluated to determine if centrifuges remain the preferred alternative.

2.5.7 Dewatering Building

A full layout of the Dewatering Building was developed based on Alternative 1 as shown in Figure 2-7 and Figure 2-8. The Dewatering Building houses the dewatering centrifuges, polymer feed system, and truck loading bay. The Centrate Equalization Basin is located adjacent to the Dewatering Facility to collect and equalize centrate for sidestream treatment. The high phosphorus DAF effluent liquid from the second stage DAFs will also be pumped to the Centrate Equalization Basin to be mixed for sidestream treatment. The Centrate Equalization Basin will be sized for 3 days of average annual centrate flow equalization. Submersible mixers will be provided in the basin to keep any potential solids in suspension and reduce deposition in the basin. The Centrate Equalization Basin and associated mixer design criteria are provided in Table 2-27. Submersible pumps will transfer flow from the Centrate Equalization Basin to sidestream treatment. The sidestream treatment feed pump design criteria are provided in Table 2-28. Additionally, a pumped spray nozzle system will be provided to reduce potential foaming in the basin. Centrate will be pumped through approximately 20 nozzles mounted to the ceiling of the tank. The nozzle feed pump criteria are provided in Table 2-29

Table 2-27 Centrate Equalization Basin Design Criteria

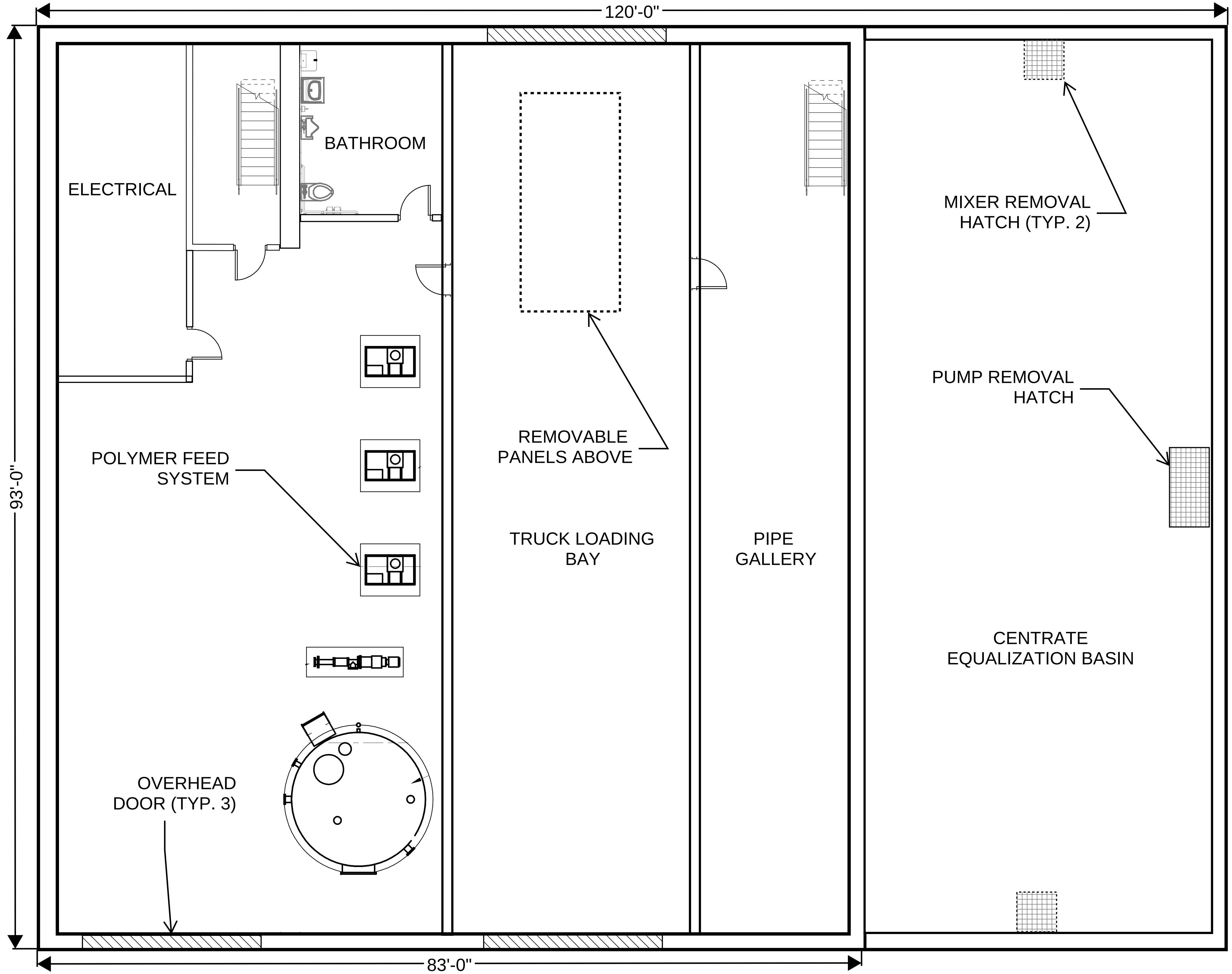
PARAMETER	DESIGN CRITERIA
Centrate Flow, gpm (Equalized)	98
Equalization time, days	3
Volume, gal	421,900
Volume, cf	56,400
Number of Mixers	2
Motor rating, hp	7.5

Table 2-28 Sidestream Treatment Feed Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Pumps	2 (1 duty, 1 standby)
Type	Submersible
Capacity, gpm (Each)	185
Motor rating, hp	10

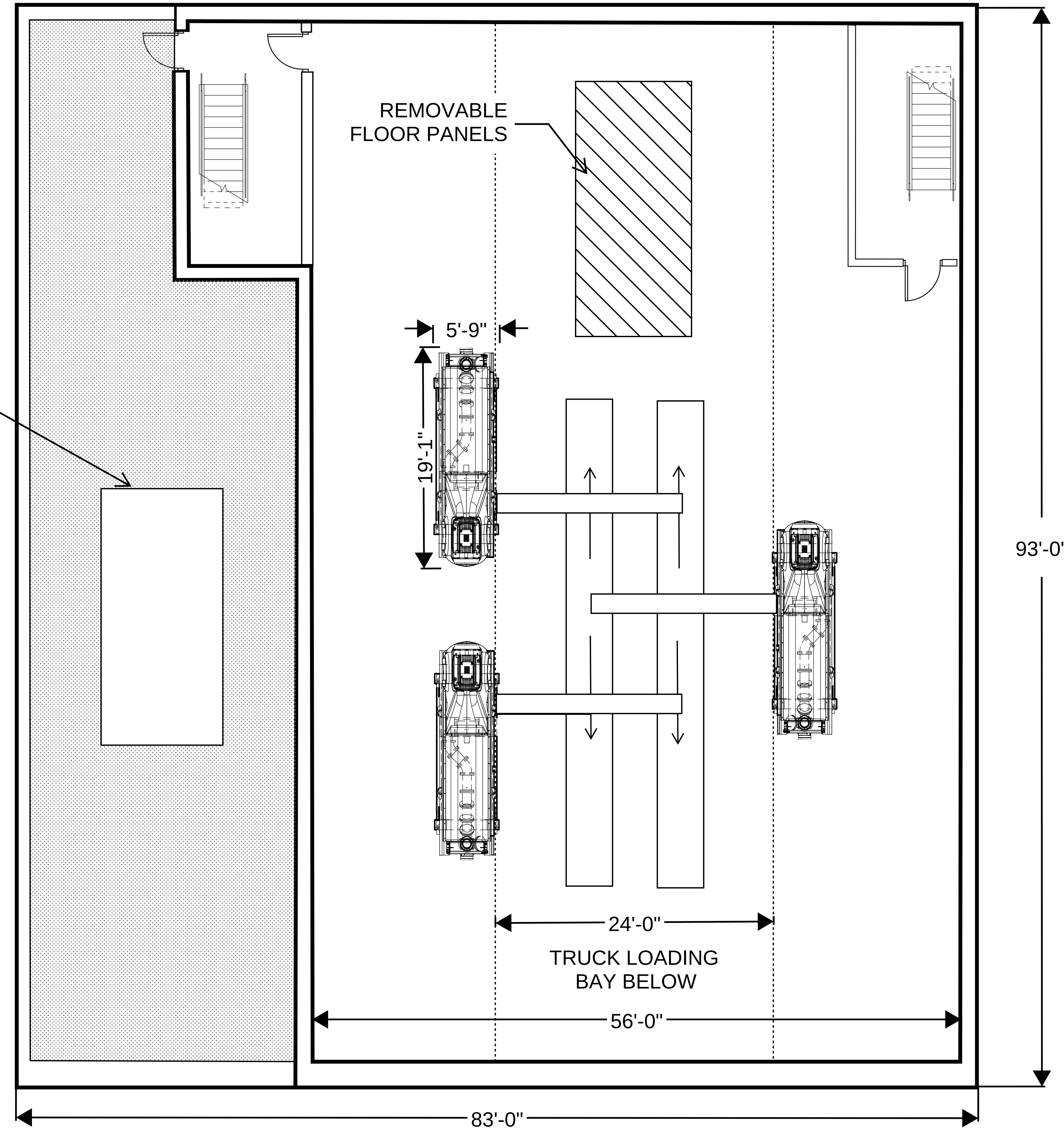
Table 2-29 Centrate Wetwell Nozzle Feed Pump

PARAMETER	DESIGN CRITERIA
Number of Pumps	1
Type	Submersible
Capacity, gpm (Each)	120
Motor rating, hp	10



JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 6 - BIOSOLIDS TREATMENT
CENTRIFUGE DEWATERING GROUND LEVEL LAYOUT
FIGURE 2 - 7



JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 6 - BIOSOLIDS TREATMENT
CENTRIFUGE DEWATERING UPPER LEVEL LAYOUT
FIGURE 2 - 8

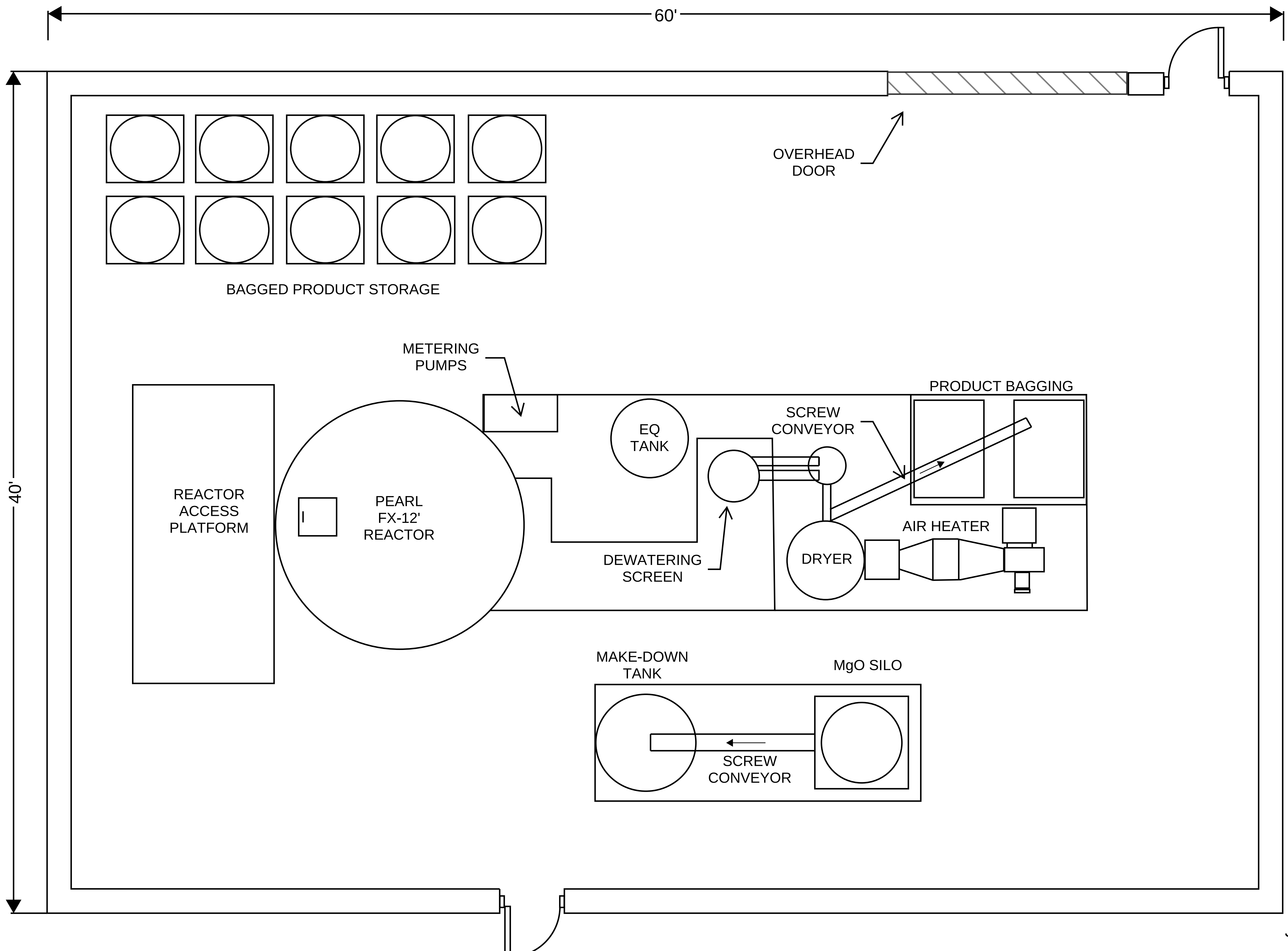
2.6 PHOSPHORUS RECOVERY

Phosphorus recovery allows for nutrients to be removed from the waste stream and turned into a useful fertilizer product. While there are several treatment technologies for phosphorus recovery, the Ostara Fx process has been selected for planning purposes in this technical memo. The Ostara process adds magnesium oxide to the sidestream, which contains high phosphorus and ammonia loads, in order to chemically precipitate struvite into granules that can be used as a fertilizer product. By removing struvite in a controlled process, the Ostara process can also reduce nuisance struvite buildup on downstream process piping and equipment. Significant nuisance struvite is not anticipated, however, and the main function of the Ostara process is anticipated to be phosphorus removal from the sidestream. The Ostara Fx process has several notable differences compared to the Ostara Pearl process that was evaluated for the THC WWTP. The Ostara Fx process equipment is smaller and has a reduced capital cost, making it a better fit for the anticipated struvite production achievable at the MCR WWTP. The Ostara Fx process also uses magnesium oxide to provide both the required magnesium and pH adjustment. The magnesium oxide is a dry powder that is diluted in water and fed as a liquid solution. The magnesium oxide feed system is provided as part of the equipment package. Design criteria for the phosphorus recovery process are provided in Table 2-30. A preliminary facility layout is shown in Figure 2-9.

The effluent from the phosphorus recovery process will be sent to the sidestream Annamox process for nitrogen removal prior to recycling the flow to the head of the plant. Additional discussion on the Annamox process is provided in TM 3 - Secondary and Sidestream Treatment.

Table 2-30 Phosphorus Recovery Design Criteria

PARAMETER	ANNUAL AVERAGE	MAXIMUM MONTH
Ostara Influent Flow, gpm	135	185
Ostara Reactor unit diameter, ft	12	12
No. of Ostara Reactor Units	1	1
Nominal PO_4-P removal, %	80	80
Struvite Produced, ppd	1,096	1,733



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JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN
TM No. 6 - Biosolids Treatment
Ostara Building Layout
FIGURE 2 - 9

2.7 DIGESTER GAS UTILIZATION

As described in paragraph 1.2.7, digester gas will be used for heating sludge for mesophilic anaerobic digestion. However, digester gas production at projected loadings is anticipated to exceed the gas required for sludge heating. The excess digester gas can be utilized as a fuel source at the MCR WWTP. For the purposes of this report, compressed natural gas (CNG) for vehicle fuel has been used for the conceptual design and cost estimate.

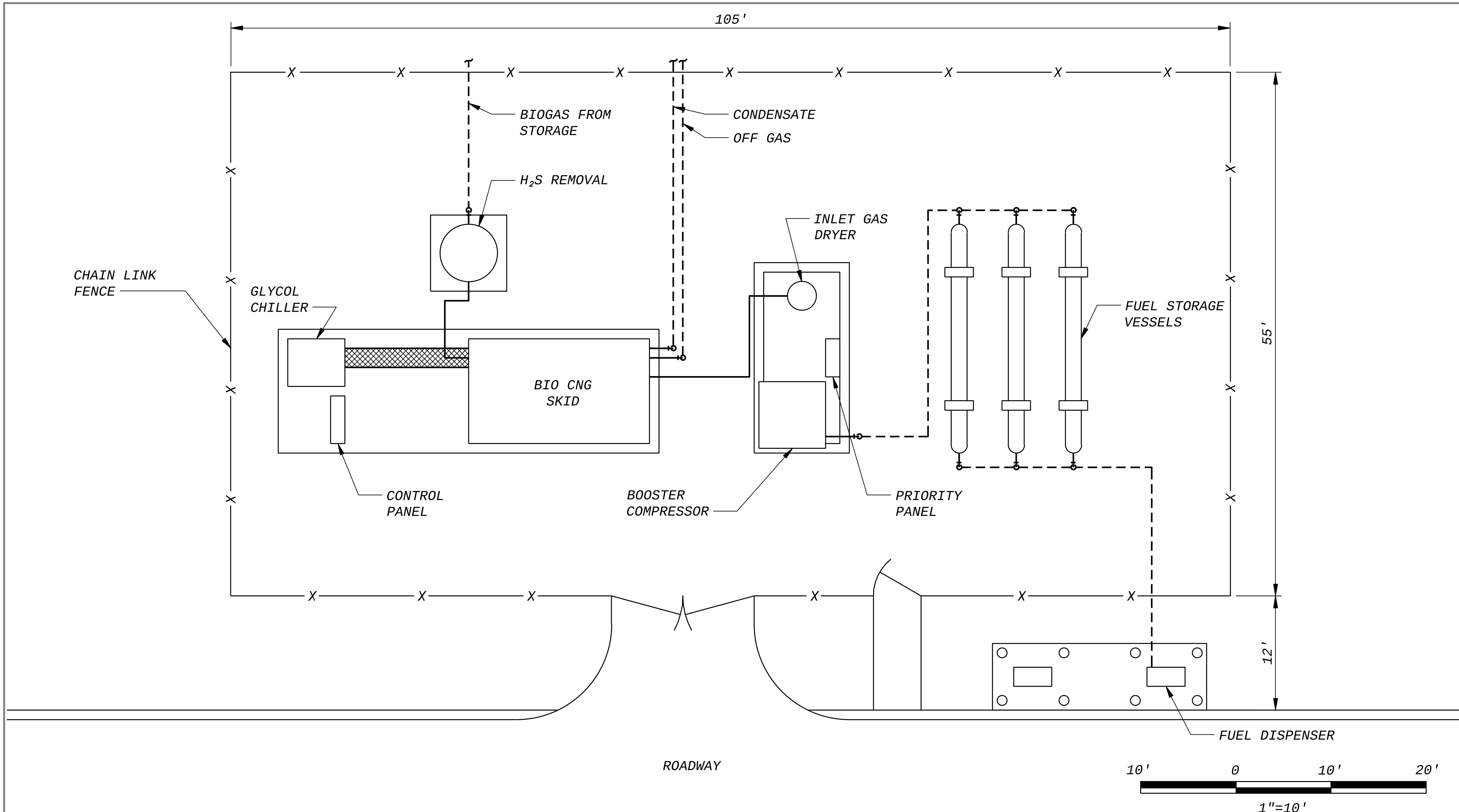
There are several different technologies for purifying digester gas and converting it to CNG. For the purposes of this report, a lower cost, lower capture efficiency membrane system has been used for the conceptual design. This system will produce CNG for vehicle fuel by removing hydrogen sulfide, siloxanes, VOC's and carbon dioxide from the biogas while compressing it for storage. Estimated CNG production is provided in Table 2-31. A layout of the CNG Facilities is provided in Figure 2-10.

Table 2-31 Digester Gas and CNG Production

PARAMETER	ANNUAL AVERAGE	MAXIMUM MONTH
Digester Gas Production, scfm	136	199
Digester Gas Heating Value, Btu/scfm	550	550
Digester Gas Production, mmBtu/day	108	158
Digester Gas to Digester Heating, mmBtu/day	63	92
Digester Gas to CNG, mmBtu/day	45	66
Digester Gas to CNG, scfm	57	83
Gas Cleaning System Capacity, scfm	100	
Methane Capture, %	65	
Fuel Conversion, Btu/GGE	114,000	
Fuel Production, GGE/day	257	376
CNG Value, $/GGE[1]	2.06	
CNG Revenue $/YR	193,238	
Estimated Vehicle Fuel Efficiency, mpg	15	
Annual Vehicle Mileage, mile/YR	1,407,000	

[1]Source: US Department of Energy Alternative Fuels Data Center October 2019 Report

CNG produced from digester gas is considered a D3 cellulosic biofuel under the EPA renewable fuel standard. Each batch of fuel is assigned a Renewable Identification Number (RIN) for EPA tracking. Each RIN has a market value that can be recovered when the fuel is sold. Additional revenue could be generated through the sale of these RIN credits. The value of RIN credits fluctuates and should be evaluated to determine if they provide an additional economic benefit if CNG production is ultimately selected for the MCR WWTP.



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JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 6 - Biosolids Treatment
CNG Facility Layout
FIGURE 2 - 10

The value of the CNG produced is expressed as revenue, however it is anticipated that the fuel would be used exclusively by JCW vehicles. Therefore, the monetary value would be realized as a reduction in the cost of fuel purchased for JCW vehicles. This report does not evaluate the fuel consumption of JCW vehicles to determine if all of the CNG produced would be utilized by JCW's fleet.

2.8 SUMMARY OF CAPITAL COSTS

The capital costs associated with each biosolids treatment facility are summarized in Table 2-31. The costs presented below do not include the cost of electrical, sitework, instrumentation and control, engineering, legal, administration (ELA), or contingencies. These costs will appear as a line item in the overall opinion of probably construction cost presented in the facility plan report.

Table 2-32 Biosolids Treatment Facilities Capital Costs

PROCESS	CAPITAL COST
Gravity Thickener/Fermenter	$2,699,000
First Stage DAF	$3,079,000
WASSTRIP Tank	$668,000
Second Stage DAF	$3,079,000
Thickening Building	$4,778,000
Digesters and Associated Equipment	$12,603,000
Digester Control Building	$10,700,000
Dewatering Building	$10,933,000
Phosphorus Recovery Building	$3,391,000
CNG Processing	$2,912,000
Total Solids Treatment Capital Costs	**$54,842,000**

- Capital costs presented in January 2020 Dollars.
- Costs exclude electrical, site, I&C and contingencies.
- Presented capital costs are conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

The MCR WWTP Gravity Thickener was modeled after the THC WWTP Gravity Thickener. A similar gravity thickener calculation was used to help build costs for other biosolids treatment facilities such as the DAF basins and WASSTRIP Tank. The Thickening, Dewatering, Digester Control and Phosphorus Recovery Buildings were laid out around the respective process equipment described in Section 2.0. The capital cost for these buildings was developed by adding up-to-date equipment quotes (including ancillary equipment and piping) to building footprint costs obtained by applying a unit price per square foot calculated from buildings of similar complexity at the THC WWTP.

The capital cost associated with the Digesters was determined by scaling digester volume from a recent BV project in Springfield, MO, as the digester complex at the THC WWTP are being retrofitted rather than replaced. Digester equipment costs, however, were scaled from the THC

WWTP. As described in Section 1.2.7, CNG was evaluated for the THC WWTP, but not implemented. The capital cost for CNG at the THC WWTP was applied to the MCR WWTP.

2.9 SUMMARY OF OPERATIONAL AND MAINTENANCE COSTS

O&M costs include the cost of power, chemicals, operating labor, general equipment maintenance, and sludge cake disposal. Revenues for sale of phosphorus fertilizer (Crystal Green) and CNG fuel are also included in the O&M Cost. O&M costs are calculated based on annual average conditions and solids production. The estimates are in January 2020 dollars.

Table 2-33 Biosolids Treatment Facilities O&M Costs

PROCESS	O&M COST
Power	$255,000
Labor	$146,000
Maintenance	$286,000
Chemicals	$322,000
Cake Disposal	$370,000
Phosphorus Revenue	($16,000)
CNG Revenue	($193,000)
Total Biosolids Treatment O&M Costs	**$1,170,000**

- Costs presented in January 2020 dollars

Annual power costs were calculated by applying a rate of $0.073/kW to the annual power consumption calculated for the equipment in each alternative. The annual labor costs associated with each alternative were calculated by estimating the number of operators and frequency of maintenance expected for each system. A rate of $33.94 was then applied to the estimated hours in order to obtain annual labor costs. Equipment maintenance was calculated as 2% of total equipment capital costs. Annual Chemical costs were calculated by applying unit costs of $1.57 per gallon of ferric chloride, $1.63 per pound of polymer, $540 per dry ton of magnesium oxide, and $3.00 per gallon of citric acid to the annual chemical use calculated for each process. Cake disposal costs were calculated by applying a unit costs of $21.00 per wet ton to the calculated annual dewatered solids production. Phosphorus revenue was calculated by applying a unit value of $75 per dry ton to the calculated annual struvite production. CNG revenue was calculated by applying a unit value of $2.06 to the calculated annual CNG production.

3.0 Summary of Findings and Recommendations

3.1 PRIMARY SLUDGE THICKENING

For primary sludge thickening and fermentation, it is recommended that two gravity thickener/fermenter tanks be constructed each at 75% capacity for flexibility and maintenance ease. Each tank will be 55-foot diameter with a flat walkable aluminum cover.

3.2 WAS THICKENING

For WAS Thickening, it is recommended that two first stage DAF thickeners, A WASSTRIP tank, and two second stage DAF thickeners be constructed to reduce the volume of WAS sent to the downstream processes and increase the phosphorus concentration for phosphorus harvesting. The first stage DAFs will be 45-foot diameter circular tanks with domed covers and will operate to reduce the WAS volume. The WASSTRIP tank will be a circular tank with a flat cover and operate to release phosphorus from the TWAS solids. The second stage DAFs will be 45-foot diameter circular tanks with domed covers and operate to separate the high phosphorus liquid from the TWAS solids. Due to the interconnectedness of these three processes, it is recommended that the associated pumps and ancillary equipment associated with each of these processes be housed in a single Thickening Building for ease of control and site layout optimization.

3.3 DIGESTION

MAD is recommended for volatile solids reduction to produce Class B sludge for land application as well as for biogas production. Four digester tanks are recommended for operation and maintenance flexibility. Digesters 1-3 should include fixed covers and serve as primary digesters, while Digester 4 should include a gas storage membrane cover and serve as a digested sludge storage tank, with the flexibility to be operated as a primary digester when one of the other tanks is taken out of service for maintenance. The digester heating should be provided by boilers utilizing biogas generated from the digestion process.

3.4 DEWATERING

Centrifuge dewatering is the recommended alternative for digested sludge dewatering. Each Centrifuge should be sized for the solids loading rate operating for approximately 7 hours per day, 5 days per week. Two redundant centrifuges should be provided due to the time required to remove the centrifuges and send them away for significant maintenance.

While centrifuges are the recommended alternative, the value of being able to perform most maintenance in-house should be re-evaluated in preliminary design in order to determine if another dewatering technology, such as belt filter presses, may be more desirable.

3.5 PHOSPHORUS RECOVERY

For phosphorus recovery, the Ostara Fx process is recommended to be constructed to remove phosphorus from the sidestream and produce a fertilizer product that can be sold. The performance of the Ostara system is linked to the WAS handling process, especially the WASSTRIP tank. Therefore, if either the WAS thickening process or the sidestream phosphorus treatment technology is changed, the other process should be re-evaluated.

3.6 DIGESTER GAS UTILIZATION

For utilization of excess digester gas, a CNG vehicle fuel production facility is recommended to be constructed. The CNG production process should be a lower-cost membrane process due to the relatively small amount of biogas produced.

DRAFT

MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 7
Support Facilities

JCW NO. MCR1-BV-17-12
B&V PROJECT 403165

PREPARED FOR

JOHNSON COUNTY KANSAS
Wastewater

OCTOBER 5, 2020

BLACK & VEATCH

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Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CANDO	Coupled Aerobic-anoxic Nitrous Decomposition Operation
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEA	Cost Effective Analyses
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
CIPP	Cured-in-place Pipe
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSBR	Continuous Sequencing Batch Reactor
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
D	
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry Weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered

Abbreviation	Meaning
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand
ffCOD	Flocculated Filtered Chemical Oxygen Demand
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
ft	Feet
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
gpcd	Gallons per capita per day
gpd	Gallons per day
gpm	Gallons per minute
H	
HB	Hallbrook Facility
HDD	Horizontal Directional Drilling
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
Hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration
IC	Internal Combustion
IFAS	Integrated Fixed-Film Activated Sludge
in	Inches
IND	Industrial
INF	Influent
IP	Intellectual Property
IPS	Influent Pump Station
IR	Irrigation Use
IRR	Irrigation
IW	Industrial Water Supply Use
J	
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDHE	Kansas Department of Health and Environment
K_e	Light Extinction Coefficient
kWh	Kilowatt-hour
L	
L	Length, Liter
lb	Pound
LF	Linear Feet
LOMR	Letter of Map Revision
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter
mgd	Million Gallons per Day
min	Minute, minimum

Abbreviation	Meaning
mJ	Millijoules
MLE	Modified Ludzack Ettinger
MLSS	Mixed Liquor Suspended Solids
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow
mmBtu	Million British Thermal Units
MOPO	Maintenance of Plant Operations
mpg	Miles per Gallon
MPN	Most Probable Number
µg/L	Micrograms per Liter
N	
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NFIP	National Flood Insurance Program
NFPA	National Fire Protection Association
NH3-N	Total Ammonia
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
NPV	Net Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow

Abbreviation	Meaning
PE	Primary Effluent
PEW	Plant Effluent Water
PFE	Primary Filtered Effluent
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow
PIF	Peak Instantaneous Flow
PLC	Programmable Logic Controller
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day
pph	Pounds per Hour
PPI	Producer Price Index
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet Weather Flow
Q	
Q	Flow
R	
RAS	Return Activated Sludge
RAS	
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
Sec	Second, Secondary

Abbreviation	Meaning
SF	Square Foot
SG	Specific Gravity
SLR	Solids Loading Rate
SMP	Stormwater Management Program, Shawnee Mission Park Pump Station
SND	Simultaneous Nitrification/ Denitrification
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station
SRT	Sludge Retention Time
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
TYP	Typical
U	
µg/L	
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VFA	Volatile Fatty Acids
VFAs	
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	Width
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation –Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WLWater LevelWK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

The purpose of this technical memorandum (TM) is to summarize the conceptual design of the support facilities at Mill Creek Regional (MCR) wastewater treatment plant (WWTP). Support facilities include Chemical Storage and Feed Equipment; Odor Control Facilities; Plant Effluent Water (PEW) Pump Station; Office, Laboratory, and Maintenance Facilities; Septage Receiving Facility, and Jet-Vac Truck Dumping Station. This TM includes a discussion of the existing support facilities, design criteria, footprint and layouts, and capital and operations and maintenance (O&M) costs.

This TM is one in a series of technical memoranda that will be incorporated into a Facility Plan report summarizing a future expansion of the MCR plant. Additional treatment processes are discussed in other TMs and site optimization of these treatment facilities are outlined in TM 8 – Site Optimization & MOPO.

1.1 BACKGROUND

Prior to this Facility Plan for MCR, an extensive alternative analysis was done for the Tomahawk Creek (THC) WWTP Expansion. The results of this analysis can be used to inform the planning of the MCR Expansion. THC WWTP is a good comparison because it is a similarly-sized facility (19 million gallons per day (mgd) annual average (AA) flow), with similar wastewater characteristics, is owned and operated by JCW, and has actual market costs for treatment technologies provided by a Contractor.

THC WWTP served as the primary comparison to develop the treatment technologies described in other MCR WWTP TMs. The Administration Building at Tomahawk serves as a useful comparison for the Office, Laboratory, and Maintenance Facilities at MCR. Additionally, the Plant Effluent Water (PEW) Pump Station at Tomahawk serves as a useful comparison for MCR WWTP. The basis of design of the Odor Control Facilities at Tomahawk was used to size the odor control systems at MCR. However, each odor control system was ultimately sized based on the proposed facilities outlined in TMs 2, 3, 4, 5, and 6.

Neither a Jet-Vac Truck Dumping Station nor Septage Receiving Facility were included in the THC WWTP Expansion. The concept for both facilities was developed based on discussions with JCW staff, existing facilities at MCR and an off-site pump station. The Chemical Storage and Feed Equipment were sized to support the treatment technologies recommended in TMs 3, 4, 6, and 8. This TM provides only a summary of the chemicals proposed for the expansion. For a supplementary explanation of each treatment process and chemicals used, refer to the respective TM.

1.2 SUMMARY OF EXISTING SUPPORT FACILITIES

1.2.1 Chemical Storage and Feed Equipment

There are currently no chemicals used in the treatment processes at MCR WWTP. One of the offsite pump stations that feeds MCR, 55th Street Pump Station, does have the capability of injecting Bioxide® for odor control purposes; however, this process is not regularly in use.

1.2.2 Odor Control Facilities

There are two different types of odor control equipment at MCR. The Influent Pumping Station (IPS) and grit removal facilities use activated carbon adsorption units to remove odorous compounds from exhaust air. The design criteria for the IPS odor control system are summarized in Table 1-1.

Table 1-1 Existing IPS Odor Control Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Adsorption Units	3
Airflow per unit, CFM	9,333
Number of Blowers	2
Blower Airflow, CFM (each)	28,000
Blower Motor, HP	100

Exhaust air from the Flow Control Structure, Grit Basins, and Grit Building is directed to an odor control system at the Grit Building. Design criteria for this system are summarized in Table 1-2.

Table 1-2 Existing Grit Chamber Odor Control Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Adsorption Units	1
Airflow per unit, CFM	4,200
Number of Blowers	2
Blower Airflow, CFM (each)	4,200
Blower Motor, HP	15

In addition, the lagoon cells have odor control mister fans. Approximately four fans service each partially mixed cell. The fans are portable and have power cords that plug into electric outlet boxes on the berms. Historically, the mister fans are used when lagoons are drained and dredged to remove solids.

1.2.3 PEW Pump Station

There is currently no Plant Effluent Water Pump Station at MCR WWTP. Service water (potable water downstream of a backflow preventer) is used for plant water needs such as washdown and seal water.

1.2.4 Office, Laboratory, and Maintenance Facilities

Currently, the main office and laboratory are located within the Operations Building at MCR WWTP. This Operations Building was constructed in 2006, and consists of a main office, control room, break room, laboratory, electrical room, janitor's closet, women's restroom, women's locker room, and a men's locker room. The Operations Building is primarily used for monitoring the plant, testing wastewater samples, and doing other office work. Separate office and maintenance facilities are located within the Maintenance/Basin Blower Building, which also includes a bathroom, shower room, and break room.

1.2.5 Septage Receiving Facility

The septage receiving facility at MCR WWTP is currently located at the IPS. The plant typically receives 50-60 haulers per week, which is distributed throughout the week - including weekends - accumulating to about 400,000 gallons per month. In addition to the current facility at MCR, JCW has septage receiving facilities at their Middle Basin and Blue River Main WWTPs. System-wide, JCW receives a total of about 1,000,000 gallons per month. It is possible that one plant would receive this amount if the remaining plants are offline for maintenance or construction.

The septage typically originates from septic tanks, commercial portable toilets, RVs, and other private uses. In 2010, B&V conducted an evaluation to assess the feasibility of incorporating nutrient removal facilities and processes at MCR. During the assessment, it was estimated that the hauled waste had an average solids concentration of two percent. Precise characteristics of the imported septage is unknown.

Since the septage receiving facility at MCR is located at the IPS, septage haulers must drive through the plant, past the lagoons and Operations Building, until they reach the IPS. The current septage receiving facility is shown in Figure 1-1.

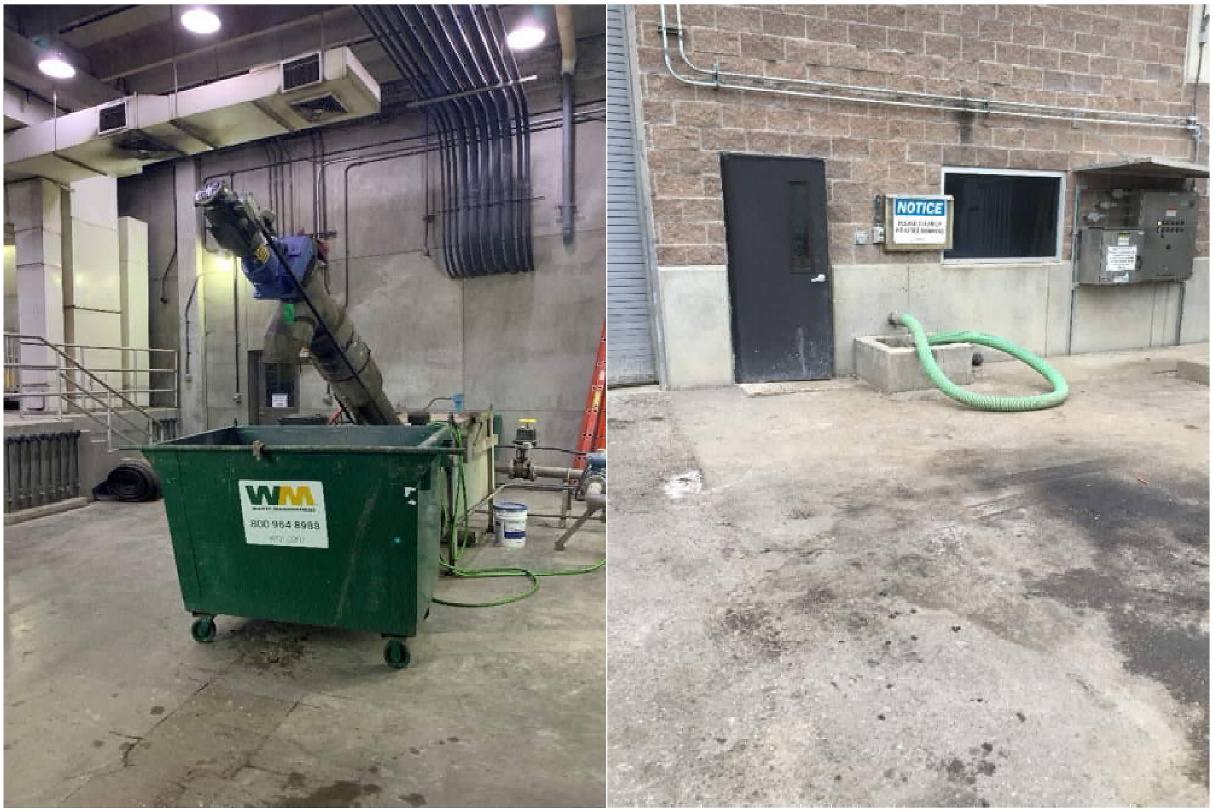


Figure 1-1 Septage Processing Unit (left) and Camlock Hose, Septage Drain, and Billing Unit (right)

The existing Septage Receiving Facility is on the northwest corner of the IPS and is equipped with a Portalogic (DS-200) billing unit. The Portalogic unit has management software capable of record keeping, billing, monitoring, and reporting. Each user is automatically billed based on their unique identification and volume of waste. This allows haulers to enter their specific identification code, connect to the camlock hose, and empty their tank.

The camlock hose is approximately four inches in diameter and 20 feet long. When not in use, the end of the hose sits in a three-foot square septage drain to prevent residual septage from leaking onto the pavement. A four-inch PVC drain at the base of the septage drain allows the washdown liquid to flow to the wet well at the IPS.

Through the hose, septage travels to the septage processing unit, a Raptor Septage Acceptance Plant (SAP) Model 31SAP by Lakeside. It has the capacity to treat 400 gallons per minute at 3 percent solids, or 285 gallons per minute at 6 percent solids. The septage that enters the SAP is washed in a 2-stage process. Screenings are captured in a washer/compactor and a quarter-inch fine screen, then are discharged from the chute.

The washed screenings discharged from the SAP are collected in a standard two cubic yard dumpster. The dumpster can be accessed via the overhead door at the northwest corner of the building and is typically emptied once weekly. The screenings are about 40 percent solids, which is approximately a 50 percent volume reduction and 67 percent weight reduction based on equipment literature. The liquid separated from the septage influent is released through an eight-inch PVC drain connected to the wet well at the IPS.

1.2.6 Jet-Vac Truck Dumping Station

The Jet-Vac Truck Dumping Station associated with MCR WWTP is currently located at the Cedar Mill Pump Station (PS). Liquid from the Jet-Vac Truck Dumping Station flows to the wet well at Cedar Mill PS and is pumped to MCR. All wastewater pumped from Cedar Mill PS enters the process on the south end of the flow control structure, which ties into the grit removal facility.

The structure consists of a 40-feet by 50-feet concrete pad sloped at two percent surrounded by a gravity block retaining wall approximately four feet tall. At the end of the sloped pad, a trench drain collects the liquid and allows it to drain to the wet well through an 8-inch PVC pipe. The current Cedar Mill PS Jet-Vac Truck Dumping Station is shown in Figure 1-2 below.

Figure 1-2 Cedar Mill PS Jet-Vac Truck Dumping Station

2.0 Design Criteria

2.1 CHEMICAL STORAGE AND FEED EQUIPMENT

Following is a summary of proposed chemical feed storage and feed facilities outlined in other TMs.

2.1.1 Supplemental Carbon

Supplemental carbon in the form of MicroC®2000, a glycerin-based carbon source, will be fed to the anaerobic and second anoxic zones of the BNR Basin. MicroC®2000 is added to the anaerobic zone to stimulate fermentation and sustain good orthophosphate (ortho-P) release. MicroC®2000 is added to the second anoxic zone for enhanced denitrification. MicroC®2000 was selected over chemicals like methanol, ethanol, and acetic acid because it is the safest of these chemicals to handle. Moreover, MicroC®2000 has greater flexibility for intermittent use compared to methanol due to the wide range of heterotrophic organisms that can utilize it as a carbon source. Leading up to design, it is recommended that an evaluation of available industrial waste sources is conducted. A waste source may be the most cost-effective supplemental carbon source, which would relegate MicroC®2000 to a back-up source in the event that industrial deliveries become unreliable.

Due to proximity of the structure to the chemical application point, the supplemental carbon system will be housed within the Basin Blower Building. For more information regarding the supplemental carbon feed system, refer to TM 3 – Secondary and Sidestream Treatment.

2.1.2 Ferric Chloride

To aid in odor control during the summer months, incoming flow to the plant will be dosed with ferric chloride. The application point will be in the collection system upstream of the IPS. Pumping modifications, including influent ferric dosing, will be described in greater detail in TM 9 – Pumping.

In the event of a biological phosphorus upset, ferric chloride will be injected into the BNR effluent upstream of the final clarifiers. This application of ferric chloride is intended to aid in effluent phosphorus removal. The ferric chloride used in the proposed BNR basin is recommended to be located at the Digester Control Building. For more information regarding ferric chloride's role in the BNR process, refer to TM 3 – Secondary and Sidestream Treatment.

Additionally, ferric chloride will be used in the biosolids treatment process to mitigate hydrogen sulfide release in the digesters. Ferric will be dosed into the sludge mixing wetwell prior to the sludge being pumped into the primary digesters. Additional ferric chloride feed points will be installed into the digested sludge lines to allow for improved dewatering and chemical P removal in the event of the WASSTRIP process going out of service for maintenance or due to an emergency situation. Moreover, if struvite becomes a problem, extra ferric chloride could be dosed at these feed points. This would mitigate struvite production but would consequently reduce the amount of phosphorus recovered in the sidestream phosphorus recovery process. Ferric chloride for the biosolids treatment processes will be stored at the Digester Control Building. For more information regarding ferric chloride's role in the biosolids treatment process, refer to TM 6 – Biosolids Treatment.

2.1.3 Micronutrients

A micronutrient system is recommended to be provided to augment essential nutrients to the sidestream deammonification process. Veolia (Kruger) will set the micronutrient formulation based upon the nutrient needs of the WWTP's biological system determined during startup. Hydrex 6913 was the chemical proposed for utilization at the THC WWTP. At the planning level, a micronutrient system is recommended, but this should be revisited during detailed design. Results from the sidestream deammonification process at THC WWTP would help guide the future micronutrient decisions for MCR WWTP.

If micronutrients are required, they would be delivered to MCR WWTP in 55-gallon drums. To prevent excess chemical handling, the micronutrient solution would be transferred into a micronutrient day tank located in the Micronutrient Feed Room of the Sidestream Deammonification Building. Data shows that plants are often able to ween off micronutrients once an Anita™ Mox system is successfully commissioned; however, results vary by plant. For more information pertaining to the Anita™ Mox and micronutrient system, refer to TM 3 – Secondary and Sidestream Treatment.

2.1.4 Sodium Hydroxide

A sodium hydroxide system will likely be provided to augment alkalinity to the sidestream deammonification process. Sodium hydroxide demand will be determined based upon the availability of alkalinity from the digesters. It is anticipated that 3,500 to 4,500 mg/L of alkalinity is required for the sidestream deammonification process, depending on the flowrate.

Historically, the digesters at the Douglas L. Smith Middle Basin (DLSMB) Treatment Plant operate at approximately 2,000-2,500 mg/L of alkalinity. Unlike what is anticipated for MCR, DLSMB accepts fats, oils, and greases (FOG) which are fed directly to the anaerobic digesters. This leads to higher rates of alkalinity consumption. Since MCR is not intended to receive FOG, the digester alkalinity concentration would likely be higher than at DLSMB, but the precise concentration is unknown. At the planning level, it is recommended a sodium hydroxide feed system is included in the sidestream deammonification process design.

THC WWTP was designed with a similar sidestream deammonification process and will have sodium hydroxide and micronutrients available to support the process. Once start-up at THC WWTP is complete, the anaerobic digester and sidestream treatment system will be closely monitored to determine the need for supplemental alkalinity. The design of the chemical feed system for the MCR sidestream treatment process should be optimized based on the experience at THC WWTP. Once in operation, the MCR digesters and sidestream treatment system should undergo similar monitoring as THC to optimize the chemical dose.

Sodium hydroxide will be stored outside the Sidestream Deammonification Building in a bulk storage tank and will be fed to the Anita™ Mox system on an as-needed basis. For more information pertaining to the Anita™ Mox and sodium hydroxide system, refer to TM 3 – Secondary and Sidestream Treatment.

2.1.5 Polymer

Polymer will be used with the centrifuges for digested sludge dewatering. The polymer storage and feed systems will be housed in the first floor of the Dewatering Building.

The WAS thickening DAFs were sized for the anticipated solids loading rate without the use of polymer; however, space was allocated in the Thickening Building layout for future polymer storage and feed equipment if needed. For more information regarding polymer's role in the biosolids treatment processes, refer to TM 6 – Biosolids Treatment.

2.1.6 Magnesium Oxide

Magnesium Oxide (MgO) is an odorless, non-hazardous chemical used in the phosphorus recovery process as both a magnesium and alkalinity source. At municipal wastewater treatment facilities, magnesium is a limiting factor in the formation of struvite; therefore, a supplementary magnesium source is required to maximize phosphorus recovery for the Ostara system. Further, pH is critical to struvite production due its effect on struvite solubility and formation of other precipitates, such as calcium carbonate. The pH range of 8 to 8.8 has demonstrated good production efficiencies and production of a high purity phosphorus product. MgO is both able to raise the pH and form struvite.

MgO is delivered as a dry powder in 1-ton bags. The Ostara Pearl Fx reactor contains a make-down system that produces a $MgO/Mg(OH)_2$ slurry from powder in a fully automated process. For additional information on the Ostara process and chemical requirements, refer to TM 6 – Biosolids Treatment.

2.1.7 Citric Acid

The Ostara Pearl Fx system design includes an acid feed system for periodic descaling of the reactor (approximately annually) and, optionally, for centrate feed lines and instrumentation upstream of the reactor through clean-in-place control loops. The frequency and dosage of the cleanings is dependent on site-specific conditions.

Any acid is suitable for descaling; however, Citric Acid (50 percent) is selected as it is safer to handle than other strong inorganic acids. It is expected less than one tote per year is needed. Storage space is allotted in the Ostara Building for Citric Acid storage. For additional information on the Ostara process and chemical requirements, refer to TM 6 – Biosolids Treatment.

2.1.8 Sodium Hypochlorite

Sodium hypochlorite will be used periodically to clean the Disk Filters. Because this is the only location where hypo is required, rather than allocating a permanent tank and pumps, it is recommended that the chemical is delivered to the site in 55-gallon drums whenever cleaning is required. For additional information on the disk filters and chemical requirements, see TM 4 – Auxiliary Wet Weather Treatment.

In addition to its permanent use, sodium hypochlorite will be required for interim disinfection during construction. Currently at MCR, the wet weather pumps at the IPS pump the wastewater to Cells 3 and 4. During construction, additional piping will be added to route wet weather flows past Cells 3, 4, 5, and 6, and deliver it directly to Cell 8. Sodium hypochlorite will be added to accelerate treatment so that the permit limits can be met with a peak flow retention time of 2.5 hours. It will be dosed at the Drop Box Structure just southeast of Cell 3 at a concentration of 10 mg/L. While this disinfection method is in place, sodium hypochlorite will be stored in a bulk storage tank near the Drop Box Structure. For additional information on construction phasing and site considerations, see TM 8 – Site Optimization and MOPO.

2.1.9 Sodium Bisulfite

Sodium bisulfite will not be used permanently at MCR, but, similar to sodium hypochlorite, it will be required for the interim disinfection treatment process during construction. Sodium bisulfite will be dosed on the south end of Cell 8 at the Plant Effluent Junction Box at a concentration of approximately 6 mg/L. The purpose of adding it to the effluent is to remove the chlorine residual before the permitted effluent sampling location at the Effluent Tunnel drop shaft inlet structure on the MCR site. Sodium bisulfite should be stored in a bulk storage tank next to the Plant Effluent

Junction Box. For additional information on construction phasing and site considerations, see TM 8 – Site Optimization and MOPO.

2.2 ODOR CONTROL

The proposed odor control systems for the facilities at MCR are activated carbon adsorption units. Similar units have shown satisfactory performance at the IPS and grit removal facilities. Additionally, activated carbon adsorption units were selected for odor control at THC WWTP. They will be radial flow units as this configuration provides a small footprint and ease of media replacement. Ductwork for the odor control systems will be stainless steel to reduce corrosion. Although this is the current preferred technology, it is possible that carbon products available today are not the same products that will be available in ten years. The carbon media should be re-evaluated and potentially updated at a date closer to project implementation.

Each odor control system will consist of two duty blowers and one standby blower for redundancy. Due to the smaller size of the odor control system for the Septage Receiving Facility and Thickening Building, only one duty and one standby blower will be used. Mist eliminators will be provided on the inlet side of each blower to remove small particulates and help prevent media plugging.

To reduce the length of ductwork between buildings and associated odor control systems, it is recommended that there be five odor control systems added total. Each odor control system will serve a cluster of related process facilities. The related processes were grouped based on their anticipated odor profile, as well as their locations in the overall plant site plan, as described in TM 8 – Site Optimization and MOPO. There will be one system for the Septage Receiving Facility; one for the existing Influent Pump Station and future Peak Flow Pump Station; one for the Headworks Building and the Primary Clarifiers; one for the Ostara, Sidestream, Dewatering, and Digester Buildings; and one for the Thickening Building, Dissolved Air Flotation (DAF) tanks, and Gravity Thickeners (GTs) / Fermenters.

For each system, foul air will be exhausted to a common header, with flow rates from individual process facilities controlled by individual adjustable and lockable dampers. Air flow rates were determined for each space based on the volume requiring odor control and air change rates per room according to NFPA 820 standards and industry practice. The air flow rates from each process are summarized in Table 2-1 below.

Table 2-1 Air Flow Rates for MCR Odor Control

Facility	Air Flow, CFM
Septage Receiving Facility	3,000
Total Septage Receiving Odor Control	**3,000**
Existing Influent Pump Station	28,000
Peak Flow Pump Station	14,050
Total Influent Pumping Odor Control	**42,050**
Primary Clarifiers	24,920
Headworks Building	17,220
Total Preliminary and Primary Treatment Odor Control	**42,140**
Sludge Wetwell	140
Dewatering Building	29,660
Ostara Building	7,200
Sidestream Reactor	920
Total Solids Processing Odor Control	**37,920**
Gravity Thickener/Fermenters	3,690
First Stage DAFs	1,680
WASSTRIP Tank	250
Second Stage DAFs	1,680
Total Thickening Odor Control	**7,300**

A dedicated odor control system will be provided for the Septage Receiving Facility because it will be located at a remote corner of the plant site to keep septage hauler traffic separate from the rest of the facility. Septage can be a significant source of odors because of its higher strength and varying composition. The design criteria for the septage receiving odor control system are provided in Table 2-2 below.

Table 2-2 Septage Receiving Odor Control Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Adsorption Units	1
Airflow per Unit, CFM	3,000
Number of Blowers	2 (1 duty and 1 standby)
Blower Airflow, CFM (each)	3,000
Blower Motor, HP	10

While the existing IPS will remain in service, a new Peak Flow Pump Station (PFPS) will be constructed to provide additional influent pumping, as described in TM 8 – Site Optimization and MOPO. The new pump station will require odor control, and it is anticipated that the activated carbon units for the existing odor control facility will need to be replaced at the time of the expansion. For planning purposes, it is assumed that the existing IPS odor control facility will be replaced with a new one that services both the existing IPS and the new PFPS.

If the Septage Receiving Facility is located in the backup location, as described in TM 8 – Site Optimization and MOPO, the influent pumping odor control system could service the Septage Receiving Facility; however, this alternative was not considered for the odor control systems described in this TM.

The total air flow presented in Table 2-1 includes the flow rates required for both influent pump stations. This combined air flow was used to determine the number and size of the future units for the combined facility. The design criteria for the influent pumping odor control system are provided in Table 2-3 below.

Table 2-3 Influent Pumping Odor Control Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Adsorption Units	3
Airflow per Unit, CFM	14,100
Number of Blowers	3 (2 duty and 1 standby)
Blower Airflow, CFM (each)	21,150
Blower Motor, HP	60

Both the Headworks Building and the Primary Clarifiers will receive odor control. At the Headworks Building, the influent and effluent box, screen room, grit room, and dumpster room will be treated for odor control. The design criteria for the preliminary and primary treatment odor control system are provided in Table 2-4.

Table 2-4 Preliminary and Primary Treatment Odor Control Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Adsorption Units	3
Airflow per Unit, CFM	14,100
Number of Blowers	3 (2 duty and 1 standby)
Blower Airflow, CFM (each)	21,150
Blower Motor, HP	60

The solids processing odor control system will control odor at the Digester, Dewatering, and Ostara Buildings. In the Digester Building, only the sludge mixing wetwell will require odor control. The Dewatering Building will receive odor control on the upper level centrifuge room, at the truck bay, and at the centrate equalization basin. The entire Ostara Building will receive odor control in addition to the sidestream treatment tanks. The design criteria for the solids processing odor control system are provided in Table 2-5 below. The solids processing areas requires slightly less airflow than the influent pumping and preliminary and primary treatment odor control areas, but the adsorption units and blowers were sized to match for the sake of consistency.

Table 2-5 Solids Processing Odor Control Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Adsorption Units	3
Airflow per Unit, CFM	14,100
Number of Blowers	3 (2 duty and 1 standby)
Blower Airflow, CFM (each)	21,150
Blower Motor, HP	60

Because the DAFs and Thickening Building are physically separated from the solids processing structures, the thickening process will have its own odor control system. The Thickening Building will not require odor control, but the Gravity Thickeners, supernatant wetwell, scum wetwell, first and second stage DAFs, and WASSTRIP will receive odor control. The design criteria for the thickening odor control system are provided in Table 2-6.

Table 2-6 Thickening Odor Control Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Adsorption Units	1
Airflow per Unit, CFM	7,300
Number of Blowers	2 (1 duty and 1 standby)
Blower Airflow, CFM (each)	7,300
Blower Motor, HP	20

In total, there will be five odor control facilities, each consisting of a concrete pad with blowers and adsorption units. The blowers will be located outdoors in sound attenuating and weather tight enclosures. In addition to these structures, it is recommended that ferric is added to the process for odor control. As described in TM 6 – Biosolids Treatment and discussed in section 2.1.2 , ferric chloride will be dosed at the sludge mixing wetwell to reduce sulfide production in the digesters. Furthermore, it is recommended that ferric chloride is dosed in the collection system. The injection point should be far enough upstream of the IPS to allow the chemical to fully mix by the time it reaches headworks, where odor control is most critical. For more information pertaining to ferric chloride in the collection system, refer to TM 9 – Pumping. Lastly, while the lagoons are online during construction, it is recommended that the odor control fan misters are left in place.

2.3 PEW PUMP STATION

To reduce the service water (potable water downstream of a backflow preventer) demand at MCR WWTP, it is recommended that a PEW Pump Station be constructed. The PEW Pump Station will be next to the UV Building and will pump water from the UV effluent channel. PEW will primarily be used for seal water, but other uses include washdown hydrants, screenings sluices, washer compactors, spray water, and centrifuge flushing. Table 2-7 shows the PEW demand used to size the system at peak and low flows for each of the structures that have PEW connections.

Table 2-7 Design PEW Demand by Structure

STRUCTURE OR AREA	DESIGN PEW DEMAND	
	Peak Flow (GPM)	Low Flow (GPM)
Influent Pump Station	160	0
Peak Flow Pump Station	130	0
Headworks	290	50
Final Sludge Pump Station	230	20
Dewatering Building	190	20
Thickening Building	190	20
Other(1)	280	50
Total	**1,470**	**160**

Notes:
(1) Total demand of all facilities with individual peak flow demands less than 100 gpm, including 2 yard hydrants.

The design criteria for the PEW Pump Station pumps are listed in Table 2-8.

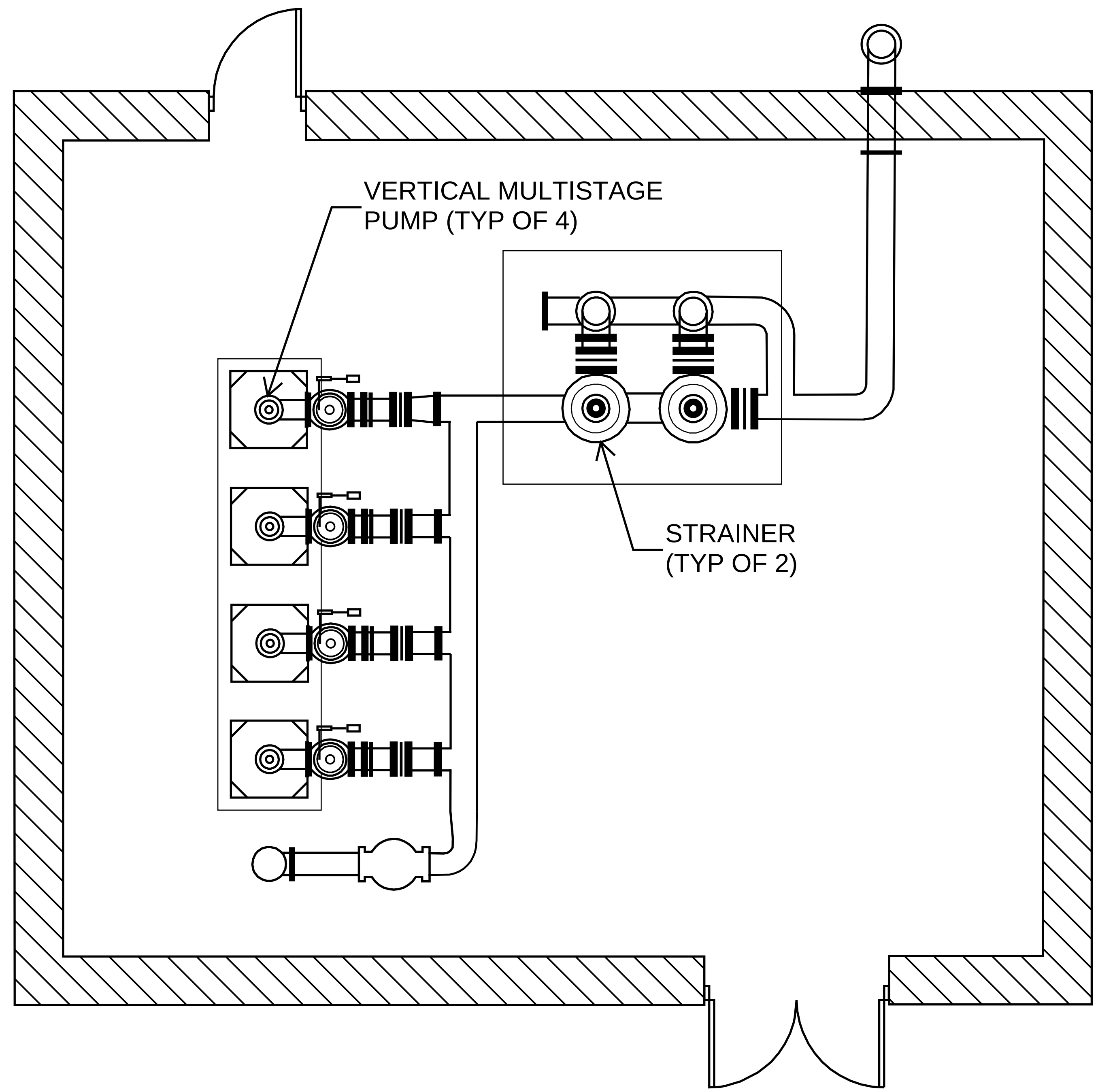
Table 2-8 PEW Pump Station Pumps Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	4 (3 duty and 1 standby)
Type	Vertical Diffusion Vane, Multistage
Drive Type	Adjustable Frequency Drive (AFD)
Pumping Capacity, each (min/max), gpm	250/600
Motor Rating, hp	60

The four pumps will have pump columns that connect to shafts that extend vertically below the floor. The impellers and bowls at the bottom of the shaft will be submerged in the UV effluent channel below and will deliver the effluent to the pump. After the UV effluent water is pumped, it will pass through straining equipment to protect downstream equipment, such as high-pressure spray nozzles, pump seals, etc.

It is not necessary to add chlorine to the PEW; however, if desired, it should be dosed at the PEW Pump Station after the strainers. Doing so would provide a chlorine residual and help prevent biofouling in the plant system. To prevent biofouling without the use of chemical, it is recommended that the system is equipped with blow-offs to allow for periodic purging of the line. This purging would be especially critical at locations that receive PEW infrequently, such as at the Gravity Thickener.

The PEW Pump Station will be an approximately 30-foot by 45-foot structure with 4 pumps and 2 automatic basket strainers. It will either be attached or directly adjacent to the UV Disinfection Facility. Electrical equipment can be stored in the electrical room in the UV Disinfection Facility. The proposed layout is shown in Figure 2-1.



SCALE 3/16" = 1'-0"

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JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

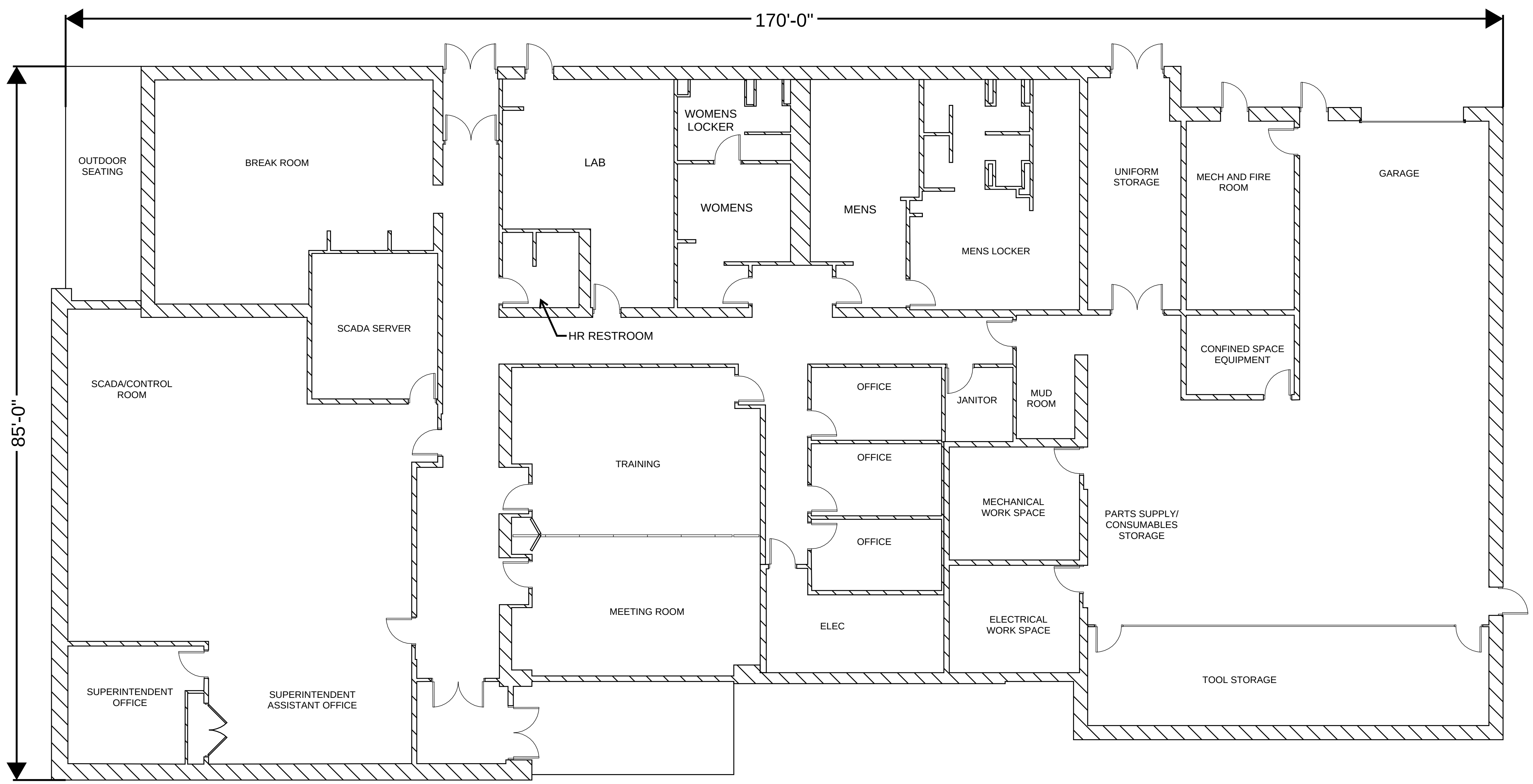
TM No. 7 - SUPPORT FACILITIES
PEW PUMP STATION PLAN
FIGURE 2 - 1

2.4 OFFICE, LABORATORY, AND MAINTENANCE FACILITIES

The current office, laboratory, and maintenance facilities at MCR WWTP are separated between the Operations Building and the Blower/Maintenance Building. The footprint of the Operations Building is about 2,170 square feet and the footprint of the office and maintenance space located inside the Blower/Maintenance Building is about 710 square feet. In comparison, at Tomahawk Creek WWTP the Administration Building, which houses all office, laboratory, and maintenance facilities, occupies a footprint of 12,140 square feet. This is approximately four times greater than the total space dedicated to office, laboratory, and maintenance facilities at MCR WWTP. Compared to just the footprint of the Operations Building, this figure is closer to five and a half times greater. Thus, rather than expanding the Operations Building, it is recommended that an entirely separate building is constructed.

For planning purposes, the MCR Administration Building was laid out identically to the one at THC WWTP. This 85 by 170-foot structure will have room for 5 offices total, including 1 for the superintendent and assistant. It will be equipped with both men's and women's restrooms, including their respective locker rooms. There will be rooms dedicated to meetings and training, as well as separate spaces for a laboratory and break room. The Administration Building also has room for maintenance, including mechanical and electrical workspaces, storage, and a large garage area. The building layout is shown in Figure 2-2.

In addition to a new Administration Building, it is recommended that a separate Maintenance Building is constructed. Not only will MCR have more staff than at Tomahawk, but there are more pump stations associated with MCR. The Maintenance Building will be located next to the Administration Building and will be approximately one-third the size. Similar to the Administration Building, the Maintenance Building will have a garage, mechanical and electrical workspaces, a restroom, storage space, and a large work area. The proposed layout is a 55 by 85-foot structure and is shown in Figure 2-3.



1/16" = 1'-0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 7 - SUPPORT FACILITIES
ADMINISTRATION BUILDING
FIGURE 2 - 2

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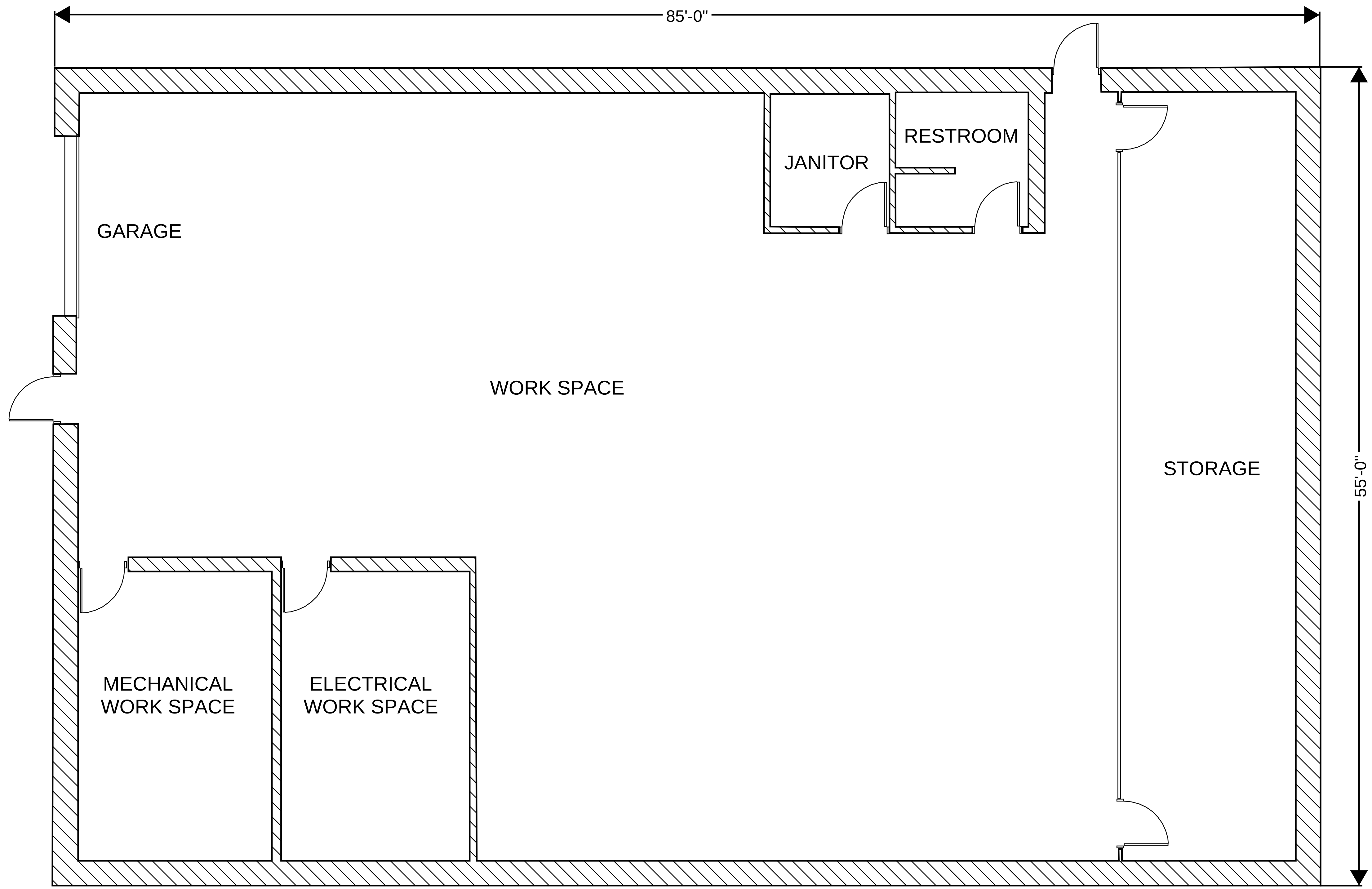

1/8" = 1'-0"

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 7 - SUPPORT FACILITIES
MAINTENANCE BUILDING
FIGURE 2 - 3

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2.5 SEPTAGE RECEIVING FACILITY

MCR typically receives 50-60 septage haulers each week, with an average weekly maximum of 12 haulers on Saturdays. If the other two JCW-owned septage receiving facilities are offline for maintenance or construction, the facility at MCR would receive septage that is typically sent to those plants. That would increase haulers up to 125-150 per week and up to 30 in a single day. In addition, MCR is not in a fully developed sewershed and could realistically see a sizeable increase in the number of local haulers in the future.

The Septage Receiving Facility should be built in a location that is easily accessible to the public and with a separate access road to prevent traffic backup. The access road should be long enough to handle a queue of 30 trucks, which is achievable given the ample space on site. At the end of the road, the pavement will fork so that vehicles will have access to either side of the facility. This configuration will minimize wait time compared to a scenario in which trucks queue in separate lines. The Septage Receiving Facility will consist of a building housing two Septage Acceptance Plants (SAPs) and accompanying equipment, a camlock hose, washdown hookups, drain, and a billing unit. There will also be extra space in the building for a third Septage Acceptance Plant to be installed in the future, if desired.

Similar to the existing Septage Receiving station, haulers will be able to enter their specific identification code into the billing unit, connect to the camlock hose, and unload their waste. Both SAPs will discharge to a dumpster, which can be accessed via an overhead door. Depending on the location of the Septage Receiving Facility on-site, the liquid discharged from the septage processing units will either flow by gravity to the IPS or will be pumped to the Headworks Building. For planning purposes, the layout includes a small wet well consisting of a prefabricated manhole and two submersible chopper pumps.

It is recommended that the SAPs be similar to the existing model at MCR, but it is recommended that they are installed with an external rock trap. The external rock trap is crucial for protecting the system since haulers bring in a wide variety of waste that may include coarse heavy materials. For example, septage collected from septic systems with a cesspool base oftentimes includes rocks. Not only will the rock trap protect the Septage Acceptance Plants, but it will protect the pumps which might otherwise need a grinder to breakdown the septage.

The Portalogic DS-200 billing unit at MCR and other JCW WWTPs has proven to be reliable, secure, and user-friendly. Its software is designed to interface with existing valves and meters. It has multiple access methods for users including keypad entry, swipe cards, key fobs, and others. It is outdoor rated, has a NEMA 4X stainless steel enclosure, and has a lockable access door. The Portalogic DS-200 also has optional packages for solar power, cold/hot climates, and gate/door control. Given JCW's positive experience with this unit, it is recommended that the new Septage Receiving Facility use the same technology.

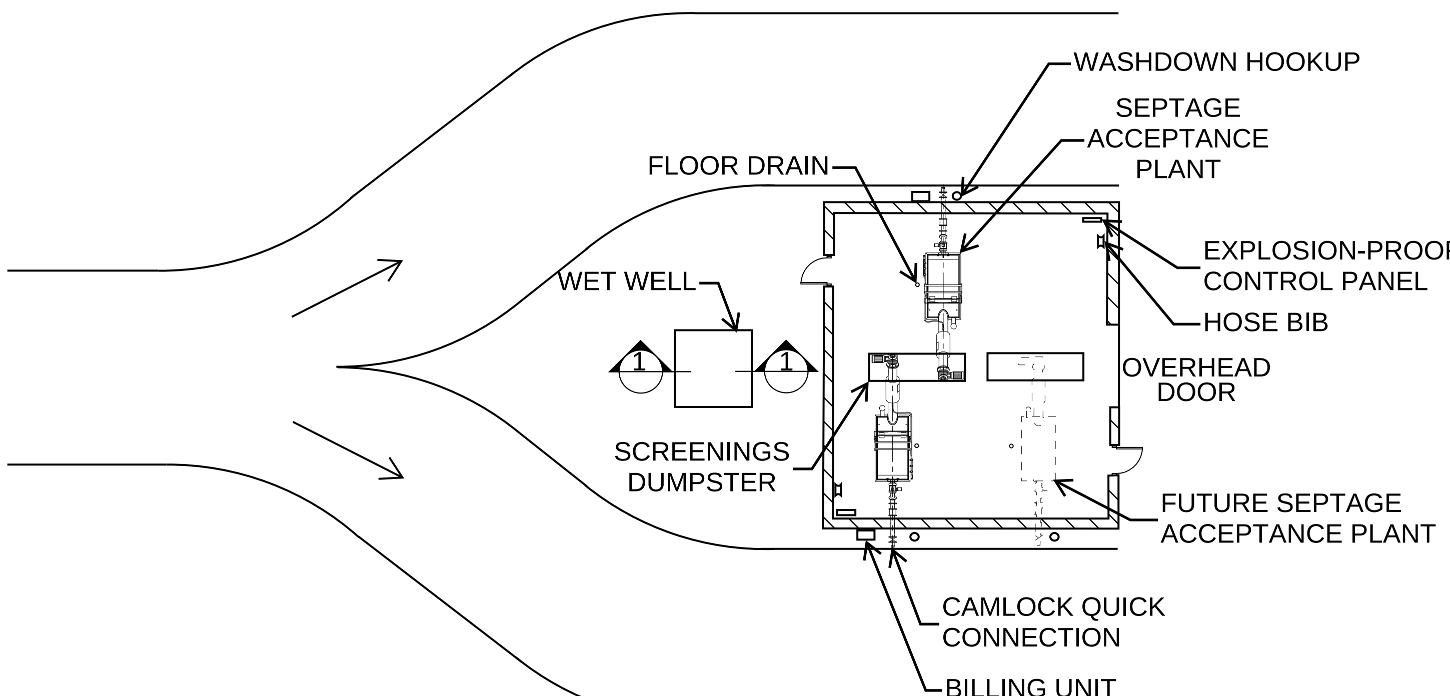
Finally, it is important to consider options for controlling the contents of hauled waste, which will ultimately be fed to the process. There is risk of high-strength and/or waste with inhibitory compounds causing adverse impacts to effluent quality, the microbial community, and human health. The most reliable method to prevent an upset would be to store incoming septage and characterize influent from every vehicle. If the sample does not meet applicable standards, the waste would be returned to the vehicle and hauled offsite. This scenario would reliably prevent a plant upset; however, it is impractical for MCR.

JCW currently owns and operates Septage Receiving Facilities at MCR WWTP, Middle Basin WWTP, and Blue River WWTP. Of these three septage receiving facilities, none employ this type of system. This type of sampling protocol would require either automated sampling equipment or a rigorous manual sampling procedure.

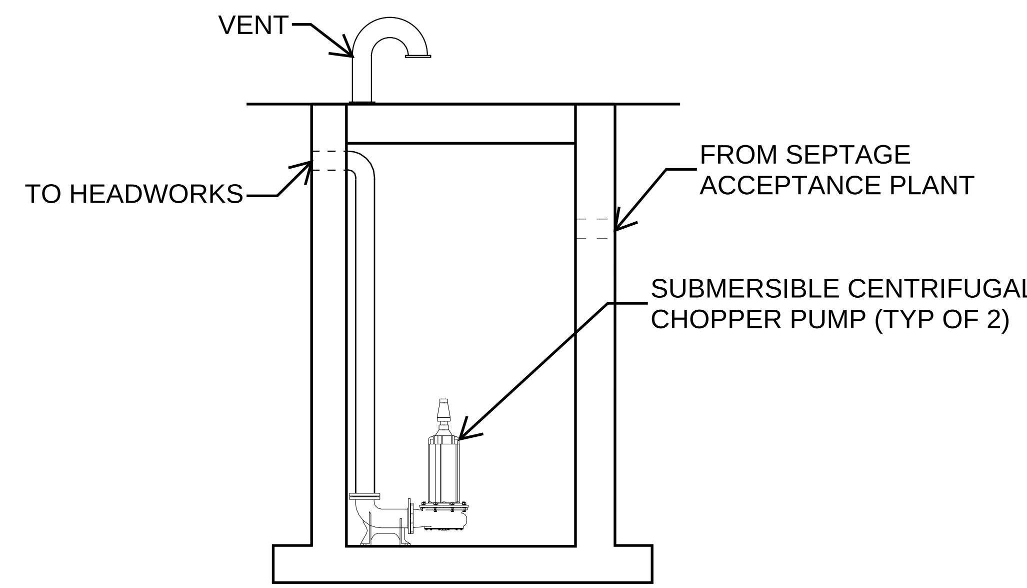
Automated sampling is possible; however, it is not practical. Due to the variability of flow from the septage haulers, any additional port could clog easily and either temporarily or permanently damage the sampler. Furthermore, introducing any new mechanical element also introduces an additional component to troubleshoot and repair. To conduct manual sampling, an equalization basin would be required to store the waste and an operator would be staffed full-time at the facility. This would not only incur additional O&M costs for staffing, but would limit septage hauling to operating hours.

Moreover, MCR WWTP currently receives about 400,000 gallons of septage per month, and system-wide JCW receives approximately 1,000,000 gallons per month. Compared to a daily influent flow rate of 21 mgd, the average septage loading at MCR is less than 0.1 percent by volume. If MCR were to receive system-wide waste, the septage loading would be less than 0.2 percent. Maintaining an equalization basin and rigid sampling protocol for such a small percentage of the flow is impractical and is not recommended for MCR WWTP.

For these reasons, it is recommended that the septage influent is manually sampled on a random or periodic basis, similar to the sampling that is already conducted at the plant. Since the waste will not be sampled after every hauler, it would not be necessary to store the septage in a tank before it is sent to another part of the plant. The layout of the Septage Receiving Facility is shown in Figure 2-4.



SEPTAGE RECEIVING PLAN
1" = 15'-0"



1 WET WELL SECTION
1" = 5'-0"

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JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 7 - SUPPORT FACILITIES
SEPTAGE RECEIVING FACILITY PLAN & SECTION
FIGURE 2 - 4

Hauled waste can be fed at multiple locations within the plant process, including the IPS, headworks, digesters (i.e., directly or through gravity thickeners), or centrifuges. Feeding septage to the digesters is not recommended due to the risk of toxic or high strength waste upsetting the process. If the septage originates from well-maintained septic systems with adequate residence times for full digestion, the waste can be sent directly to dewatering and efficiently processed through the centrifuges; however, this is not the case for all septic systems and MCR is likely to receive hauled waste from a variety of sources. Most hauled waste will require additional treatment. In that case, the waste can be sent to the IPS or directly to headworks. The MCR septage receiving facility design uses the Lakeside Septage Acceptance Plant (Model 31SAP) to screen hauled waste through 1/4" screens prior to introduction with the mainstream. Therefore, the hauled sludge can bypass the IPS bar screens and be introduced at the Headworks Building upstream of the fine screens, which is recommended for MCR WWTP.

2.6 JET-VAC TRUCK DUMPING STATION

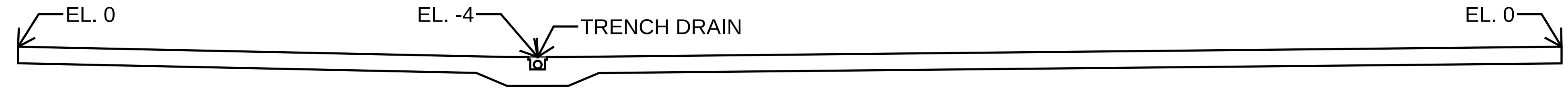
Jet-Vac trucks are owned and operated by JCW and are currently being dumped at Cedar Mill Pump Station. This pump station is relatively close to MCR WWTP; however, pumping at Cedar Mill is still required to get the waste to MCR. In the future, it would be more convenient for JCW to be able to have a Jet-Vac receiving station at MCR. There is also a possibility that a Jet-Vac receiving station could be incorporated into the Nelson WWTP design. Either location would allow the existing Jet-Vac receiving at Cedar Mill to be abandoned at some point in the future. This is preferred because Cedar Mill PS is located near a growing residential area, which is not a good location for the Jet-Vac truck traffic. This TM provides details for including a Jet-Vac receiving station at MCR.

Unlike the Septage Receiving Facility, the Jet-Vac Truck Dumping Station will only receive waste from JCW-owned trucks. Thus, there is no need to consider the risks associated with private haulers or to locate it outside of the plant gates. The biggest consideration for locating the Jet-Vac dumping station is odor control since it is completely exposed and will not have a complementary odor control facility. Secondarily, the Jet-Vac trucks should also be able to access it without added difficulty. Ideally, the dumping station would be located near the point where it will be introduced to the plant stream to reduce piping and pumping costs.

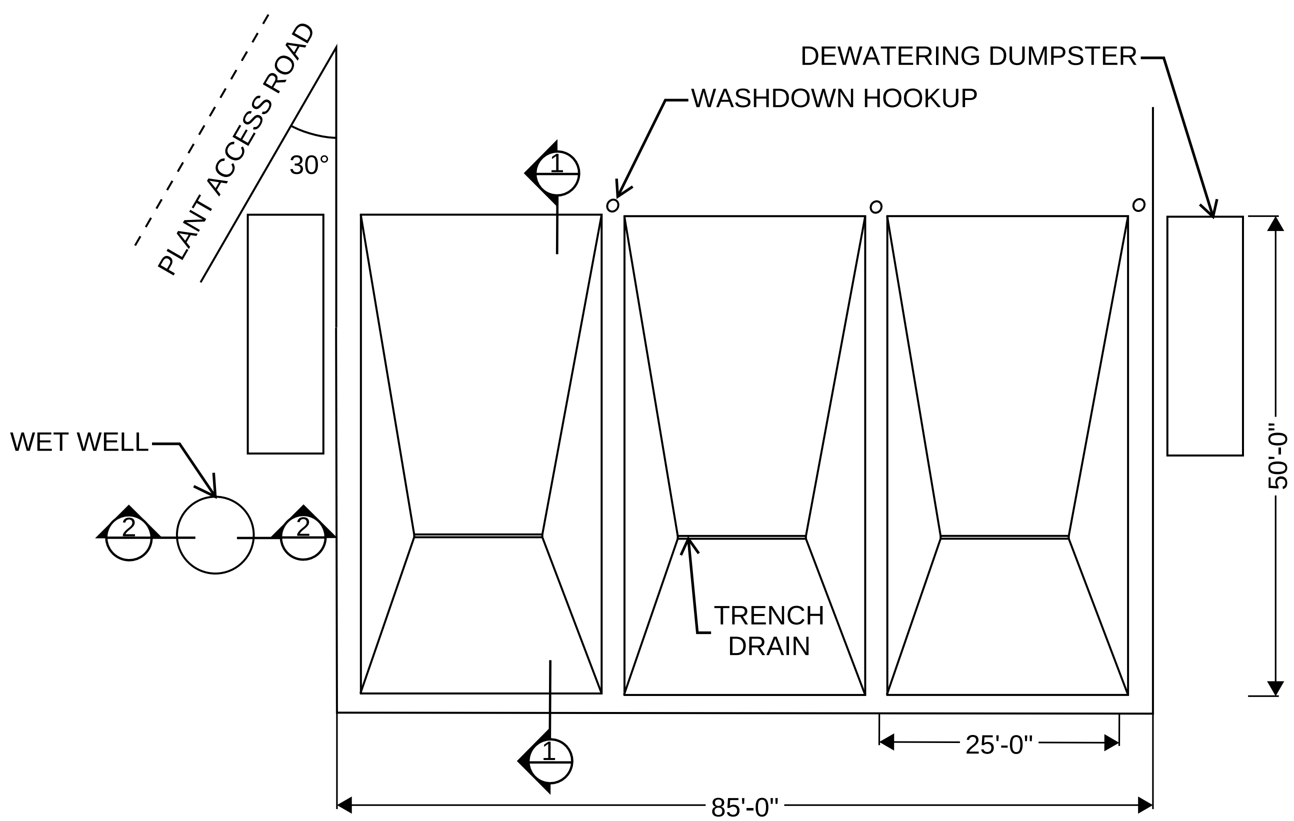
To facilitate access for the Jet-Vac trucks, the concrete pad will be situated at a 30-degree angle relative to the road. Trucks will be able to drive slightly past the driveway, back in easily, then end dump the load and exit the plant. This design eliminates turns or other maneuvers that would make the facility less safe and more difficult to access.

The Jet-Vac Truck Dumping Station will consist of three separate bays to allow for different phases of operation – decanting, solids discharge, and truck rinsing. Each bay will be a 25 by 50-foot sloped concrete pad with trench drain, and each will be surrounded by a 4-foot containment wall to prevent spills. The dumping station will be outfitted with lights, washdown hookups for cleaning, and two 25 cubic yard dewatering dumpsters.

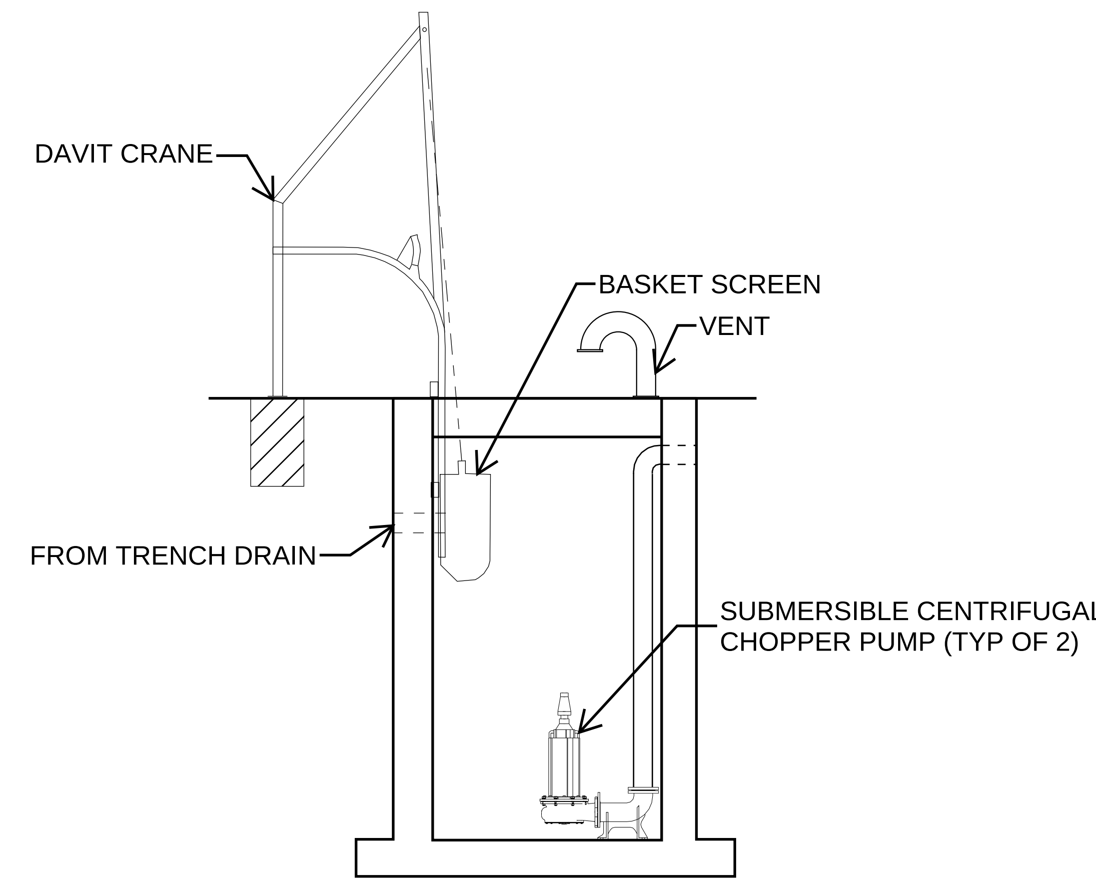
Each of the 3 bays was sized to handle the capacity of 1 full-sized Jet-Vac truck based on a minimum width of 25 feet, maximum slope of 1 percent, and maximum liquid surface depth of 2 inches. The total area of the dumping station was minimized in order to mitigate the amount of runoff collected in the wet well during storm events. From the wet well, the liquid could either be pumped to the Headworks Building or flow by gravity to the Influent Pump Station. For planning purposes, a wet well equipped with two submersible chopper pumps and a basket screen with davit crane was considered. The proposed layout of the Jet-Vac Truck Dumping Station is shown in Figure 2-5.



1 TRENCH DRAIN SECTION
1" = 5'-0"



JET-VAC PLAN
1" = 20'-0"



2 WET WELL SECTION
1" = 5'-0"

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JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 7 - SUPPORT FACILITIES
JET-VAC PLAN & SECTION
FIGURE 2 - 5

3.0 Cost Analysis

Preliminary capital and O&M costs were developed for the support facilities as described in Section 2.0. The costs presented below do not include the cost of electrical; sitework; instrumentation and control; engineering, legal, administration (ELA); or contingencies. These costs will appear as line items in the overall opinion of probable construction cost presented in TM 10 - Implementation. The estimates are in 2020 dollars.

3.1 SUMMARY OF CAPITAL COSTS

3.1.1 Chemical Storage and Feed Equipment

All capital costs associated with Chemical Storage and Feed Equipment are incorporated in the overall capital costs of the treatment processes they supplement. These processes and capital costs are presented in TMs 3, 4, 6, and 8.

3.1.2 Odor Control Capital Costs

The estimated capital costs for the odor control facilities are summarized in Table 3-1 below. Each odor control facility cost includes carbon adsorption units, blowers, sound attenuation units, and all other associated pipes, valves, and equipment.

Table 3-1 Odor Control Capital Costs

Process	Capital Cost
Septage Receiving Odor Control	$140,000
Influent Pumping Odor Control	$1,963,000
Preliminary and Primary Treatment Odor Control	$1,973,000
Solids Processing Odor Control	$1,777,000
Thickening Odor Control	$345,000
Total	**$6,198,000**

- Capital costs presented in 2020 Dollars.
- Costs exclude electrical, site, I&C, ELA and contingencies.
- Presented capital costs are at a conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

3.1.3 PEW Pump Station

Estimated costs for the PEW Pump Station are presented in Table 3-2. These costs include equipment, piping and accessories, and building costs.

Table 3-2 PEW Pump Station Capital Cost

	CAPITAL COST
PEW Pump Station	$1,038,000

- Capital costs presented in 2020 Dollars.
- Costs exclude electrical, site, I&C, ELA and contingencies.
- Presented capital costs are conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

3.1.4 Office, Laboratory, and Maintenance Facilities

Estimated costs for the Administration Building and Maintenance Building, which will house all office, laboratory, and maintenance facilities at MCR WWTP, are presented in Table 3-3. These estimates include building costs such as HVAC, plumbing, and electrical appurtenances.

Table 3-3 Office, Laboratory, and Maintenance Facilities Capital Cost

	CAPITAL COST
Administration Building	$4,054,000
Maintenance Building	$1,351,000
Total	**$5,405,000**

- Capital costs presented in 2020 Dollars.
- Costs exclude electrical, site, I&C, ELA and contingencies.
- Presented capital costs are at a conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

3.1.5 Septage Receiving Facility

The estimated capital cost for the Septage Receiving Facility is presented in Table 3-4 below. This estimate includes the cost of the septage receiving equipment associated with two stations, overall building costs, and the wet well structure (including pumps). It does not include the additional pavement associated with the road used to access the facility or the equipment associated with a third septage receiving station.

Table 3-4 Septage Receiving Facility Capital Cost

	CAPITAL COST
Septage Receiving Facility	$867,000

- Capital costs presented in 2020 Dollars.
- Costs exclude electrical, site, I&C, ELA and contingencies.
- Presented capital costs are at a conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

3.1.6 Jet-Vac Truck Dumping Station

The estimated capital cost for the Jet-Vac Truck Dumping Station is presented in Table 3-5 below. This estimate includes the cost of the concrete pad, retaining wall, and wet well structure (including pumps, basket screen, and davit crane).

Table 3-5 Jet-Vac Truck Dumping Station Capital Cost

	CAPITAL COST
Jet-Vac Truck Dumping Station	$303,000

- Capital costs presented in 2020 Dollars.
- Costs exclude electrical, site, I&C, ELA and contingencies.
- Presented capital costs are at a conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high).

3.2 SUMMARY OF OPERATIONAL AND MAINTENANCE COSTS

The design criteria and manufacturer data were used for equipment sizing and ultimately the applicable power consumption requirements. Power O&M costs were estimated based on a rate of $0.073 kWh. The equipment labor costs are based on a B&V estimate of hours per week of total labor associated with the support facility and an hourly rate of $33.94. The equipment maintenance cost is assumed to be 2 percent of the equipment capital cost.

3.2.1 Chemical Storage and Feed Equipment

All O&M costs associated with Chemical Storage and Feed Equipment are incorporated in the overall O&M costs of the treatment processes they supplement. These processes and O&M costs are presented in TMs 3, 4, 6, and 8.

3.2.2 Odor Control Facilities

The operational and maintenance costs for the odor control systems consist primarily of the electrical cost for running the dedicated odor control blowers and the cost of replacing exhausted media. While there are routine maintenance tasks associated with the odor control system - such as cleaning the mist eliminators and lubricating the blowers - these tasks are infrequent, resulting in a minimal labor cost associated with odor control. The equipment maintenance cost includes the cost of maintaining the blowers and periodic carbon media replacement. O&M costs developed for odor control facilities are summarized in Table 3-6.

Table 3-6 Odor Control O&M Costs

	ANNUAL O&M COST
Power	$167,000
Labor	$2,000
Equipment Maintenance	$25,000
Media Replacement	$610,000
Chemicals	-
Total Annual O&M Costs	**$804,000**

3.2.3 PEW Pump Station

O&M Costs for the PEW Pump Station are presented in Table 3-7. These costs are primarily associated with the operation of the vertical diffusion vane pumps.

Table 3-7 PEW Pump Station O&M Costs

	ANNUAL O&M COST
Power	$86,000
Labor	$4,000
Equipment Maintenance	$5,000
Chemicals	-
Total Annual O&M Costs	**$95,000**

3.2.4 Office, Laboratory, and Maintenance Facilities

O&M costs for the Administration Building and Maintenance Building include building power (lighting and HVAC) and labor costs for full-time staff. These costs are wrapped up in the overall plant O&M cost presented in TM 10 – Implementation. There will likely be power and equipment maintenance costs for laboratory equipment, but since the equipment is so small, these costs are negligible.

3.2.5 Septage Receiving Facility

The primary O&M cost of the Septage Receiving Facility is related to operation and maintenance of the septage processing equipment. O&M costs developed for the Septage Receiving Facility are summarized in Table 3-8.

Table 3-8 Septage Receiving Facility O&M Costs

	ANNUAL O&M COST
Power	$1,000
Labor	$2,000
Equipment Maintenance	$8,000
Chemicals	-
Total Annual O&M Costs	**$11,000**

3.2.6 Jet-Vac Truck Dumping Station

The power and equipment maintenance costs are associated with the two chopper pumps. Additional labor is required to remove the solid debris from the bays. O&M costs developed for the Jet-Vac Truck Dumping Station are summarized in Table 3-9.

Table 3-9 Jet-Vac Dumping Station O&M Costs

	ANNUAL O&M COST
Power	$1,000
Labor	$7,000
Equipment Maintenance	$1,000
Chemicals	-
Total Annual O&M Costs	**$9,000**

4.0 Summary of Findings and Recommendations

4.1 CHEMICAL STORAGE AND FEED EQUIPMENT

Supplemental carbon will be fed to the BNR basins and stored in the Basin Blower Building. Ferric chloride will be dosed in the collection system upstream of the IPS, at the final clarifiers, and at the digesters. The ferric dosed in the collection system will be stored at the Peak Flow Pump Station. Ferric dosed at the final clarifiers and the digesters will be stored at the Digester Control Building. Micronutrients and sodium hydroxide will be supplied to the sidestream deammonification process. Micronutrients will be stored in the Micronutrient Feed Room while sodium hydroxide will be stored outside of the Sidestream Deammonification Building. Polymer supplied to the centrifuges will be stored on the first floor of the Dewatering Building. If polymer is used in the future at the DAFs, it will be stored in the Thickening Building. Sodium hypochlorite and sodium bisulfite used for interim disinfection during construction will be temporarily stored at the Drop Box Structure and Plant Effluent Junction Box, respectively. Sodium hypochlorite used for cleaning of the Disk Filters will not have a permanent storage location but will be delivered to the site when needed for cleaning.

4.2 ODOR CONTROL FACILITIES

It would be optimal to add five odor control facilities, given the relative locations of the proposed facilities on site. These systems will be added to mitigate odors at each of the separate clusters of structures, while minimizing the amount of ductwork required. These systems will service the Septage Receiving Facility, Influent Pump Stations, Preliminary and Primary Treatment, Solids Processing, and Thickening areas. The odor control facilities will utilize activated carbon adsorption units and two to three blowers to neutralize odors. In addition to the odor control facilities, ferric chloride will be dosed in two locations. It will be injected in the collection system to mitigate odors at the Headworks Building and will be added to the sludge mixing wetwell to reduce sulfide production in the digesters. It is recommended that the odor control misting fans remain in service as long as the lagoon cells remain online.

4.3 PEW PUMP STATION

The PEW Pump Station will be a 30-foot by 45-foot structure attached to the UV Disinfection Facility. It will be outfitted with four vertical diffusion vane multistage pumps and two automatic strainers. The pumps will have vertical suction pipes submerged in the UV effluent channel below. All PEW will pass through the strainers before being distributed to the structures on site with PEW service.

4.4 OFFICE, LABORATORY, AND MAINTENANCE FACILITIES

Office, laboratory, and maintenance facilities will be housed in an Administration Building and a Maintenance Building. The Administration Building will be an 85 by 170-foot structure and will include multiple offices, a laboratory, storage and maintenance spaces, a training room, meeting room, and break room. The Maintenance Building will have a footprint approximately one-third the size, sitting at 50 by 75 feet. It will have a garage, mechanical and electrical workspaces, a restroom, a storage room, and an open workspace.

4.5 SEPTAGE RECEIVING FACILITY

To handle the septage load at MCR WWTP, it is recommended that a structure is constructed with two septage receiving stations, including space for a third station to be added in the future. The facility should be accessible to the public and the access road should be configured to handle 30 haulers queued at one time without affecting traffic on the surrounding roads. On the building exterior, each station will be equipped with a Portalogic billing unit, camlock quick connection hose, washdown fixtures, and drain. Discharged screenings will be collected in a dumpster while the effluent will drain to a wet well. The wet well will be equipped with two submersible chopper pumps that will pump the liquid to the fine screens at the Headworks Building for further treatment.

4.6 JET-VAC TRUCK DUMPING STATION

As part of the MCR WWTP expansion, a Jet-Vac Receiving Station will be added to the facility. This dumping station will have 3 bays, each with a 25 by 50-foot sloped concrete pad and a trench drain. Each of the three bays will be surrounded by a four-foot containment wall. The structure will be angled at 30 degrees relative to the road to improve ease of access and will be wide enough to accommodate 3 JCW Jet-Vac trucks and two dewatering dumpsters. Drained liquid will be collected in a wetwell equipped with a basket screen and two submersible chopper pumps. From the Jet-Vac Receiving Station, effluent will be sent to the Influent Pump Station.

DRAFT

MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 8
Site Optimization and MOPO

JCW NO. MCR1-BV-17-12
B&V PROJECT 403165

PREPARED FOR

JOHNSON COUNTY KANSAS
Wastewater

OCTOBER 5, 2020

BLACK & VEATCH

Table of Contents

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LIST OF FIGURES

Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
APWA	American Public Works Association
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CANDO	Coupled Aerobic-anoxic Nitrous Decomposition Operation
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEA	Cost Effective Analyses
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
CIPP	Cured-in-place Pipe
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSBR	Continuous Sequencing Batch Reactor
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
cy	Cubic Yard
D	
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry Weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
FC	Final Clarifier
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered

Abbreviation	Meaning
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand
ffCOD	Flocculated Filtered Chemical Oxygen Demand
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
FPS	Feet per Second
ft	Feet
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
gpd/sf	Gallons per Day per Square Foot
gpm	Gallons per Minute
H	
HB	Hallbrook Facility
HDD	Horizontal Directional Drilling
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration

Abbreviation	Meaning
IC	Internal Combustion
IFAS	Integrated Fixed-Film Activated Sludge
in	Inches
IND	Industrial
INF	Influent
IP	Intellectual Property
IPS	Influent Pump Station
IR	Irrigation Use
IRR	Irrigation
IW	Industrial Water Supply Use
J	
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDHE	Kansas Department of Health and Environment
K_e	Light Extinction Coefficient
kWh	Kilowatt-hour
L	
L	Length, Liter
lb	Pound
LF	Linear Feet
LOMR	Letter of Map Revision
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter

Abbreviation	Meaning
mgd	Million Gallons per Day
min	Minute, minimum
mJ	Millijoules
MLE	Modified Ludzack Ettinger
MLSS	Mixed Liquor Suspended Solids
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow
mmBtu	Million British Thermal Units
MOPO	Maintenance of Plant Operations
mpg	Miles per Gallon
MPN	Most Probable Number
µg/L	Micrograms per Liter
N	
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NFIP	National Flood Insurance Program
NH3-N	Total Ammonia
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
NPV	Net Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow

Abbreviation	Meaning
PSF	Peak Secondary Flow
PE	Primary Effluent
PFE	Primary Filtered Effluent
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow
PIF	Peak Instantaneous Flow
PLC	Programmable Logic Controller
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day
pph	Pounds per Hour
PPI	Producer Price Index
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet-weather Flow
Q	
Q	Flow
#Q	# Times Q
R	
RAS	Return Activated Sludge
RAS	
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RCP	Reinforced Concrete Pipe
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
SCS	Soil Conservation Service
Sec	Second, Secondary

Abbreviation	Meaning
SF	Square Foot
SG	Specific Gravity
SLR	Solids Loading Rate
SMP	Stormwater Management Program, Shawnee Mission Park Pump Station
SND	Simultaneous Nitrification/ Denitrification
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station
SRT	Sludge Retention Time
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TR-55	Technical Release 55
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge

Abbreviation	Meaning
TYP	Typical
U	
USDA	United States Department of Agriculture
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VFA	Volatile Fatty Acids
VFAs	
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	Width
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation –Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WLWater LevelWK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

1.1 BACKGROUND

The purpose of this technical memorandum (TM) is to identify the optimal site configuration and required Maintenance of Plant Operations (MOPO) activities for the selected treatment processes for the Mill Creek Regional (MCR) Wastewater Treatment Plant (WWTP) Expansion. This will require achieving several interdependent objectives, including:

- An efficient facility layout from a wastewater operations and hydraulic perspective.
- Maintaining plant operations during construction to meet permit limits.
- Providing redundancy to critical areas to eliminate single points of failure.
- Understanding constructability and sequencing of future facilities and identifying locations to allow a streamlined construction.

This TM includes discussion of the existing site, various factors affecting the site layout, preliminary layout alternatives, and provides a recommended layout based on Black & Veatch (BV) analysis and discussion with Johnson County Wastewater (JCW). This TM is one in a series of technical memoranda for the MCR Facility Plan. Specific treatment processes, influent pumping, support facilities, site utilities, and other implementation considerations — such as permitting requirements — are discussed in other TMs. A plant process flow diagram of the selected treatment processes for the MCR WWTP Expansion is presented in Figure 1-1.

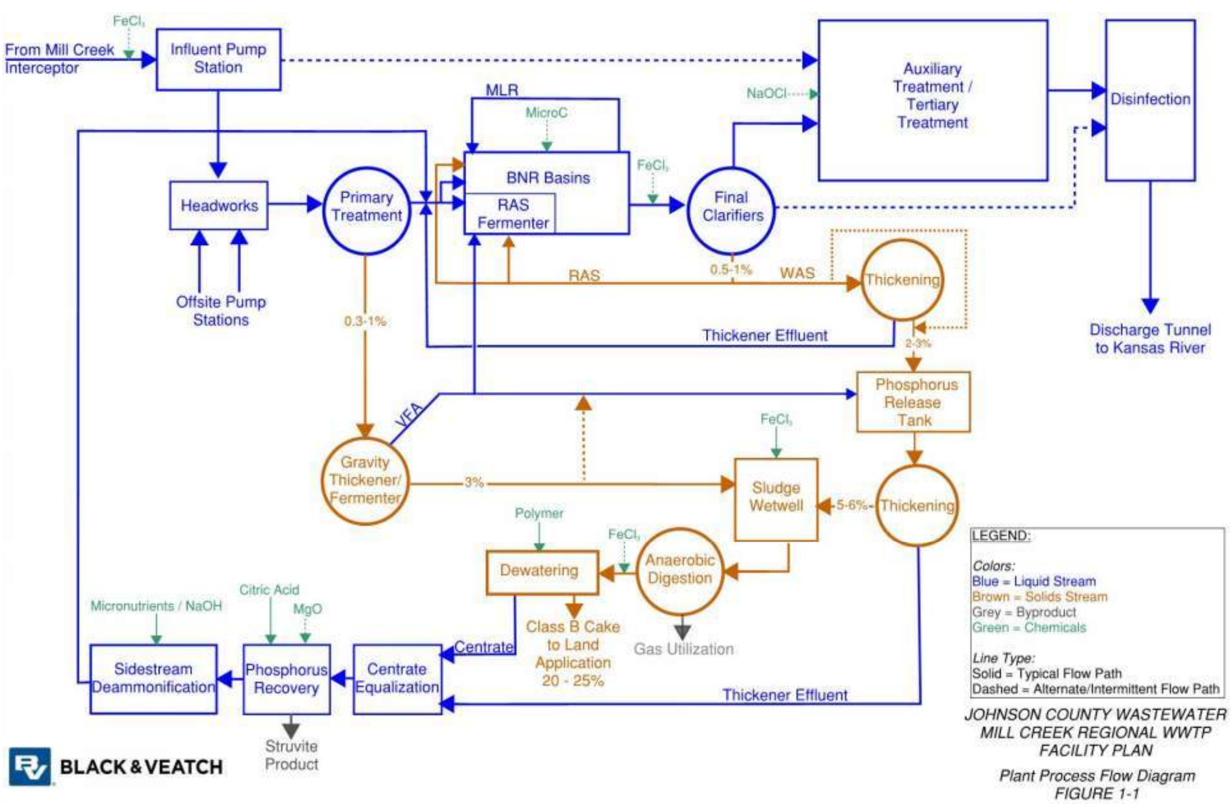


Figure 1-1 MCR Plant Process Flow Diagram

1.2 EXISTING SITE

The MCR WWTP site, shown in Figure 1-2, is located at 20001 West 47th Street, Shawnee, Kansas 66218. This is just west of Interstate 435 and just south of the Kansas River, near the confluence of Mill Creek and the Kansas River. The WWTP effluent flows through a tunnel and discharges into the Kansas River through submerged diffusers downstream of the Water One jetty.

Figure 1-2 MCR WWTP Location

The extents of the Federal Emergency Management Agency (FEMA) 100-year and 500-year floodplains are shown in Figure 1-3. Most of the MCR site is located above the 500-year floodplain elevation of approximately 779.0 feet (ft), including all existing facilities. The only area below the 500-year floodplain and above the 100-year floodplain is a small section in the northern corner of the site near the intersection with Wilder Road. The only areas below the 100-year floodplain elevation of approximately 773.0 ft, are along the southern border near the Mill Creek bank and along the eastern border along Wilder Road. The topography along these southern and eastern borders of MCR are steep enough that the locations of the 100-year and 500-year contours are nearly identical. Sufficient space is available onsite to allow all new facilities to be located above the 500-year floodplain.

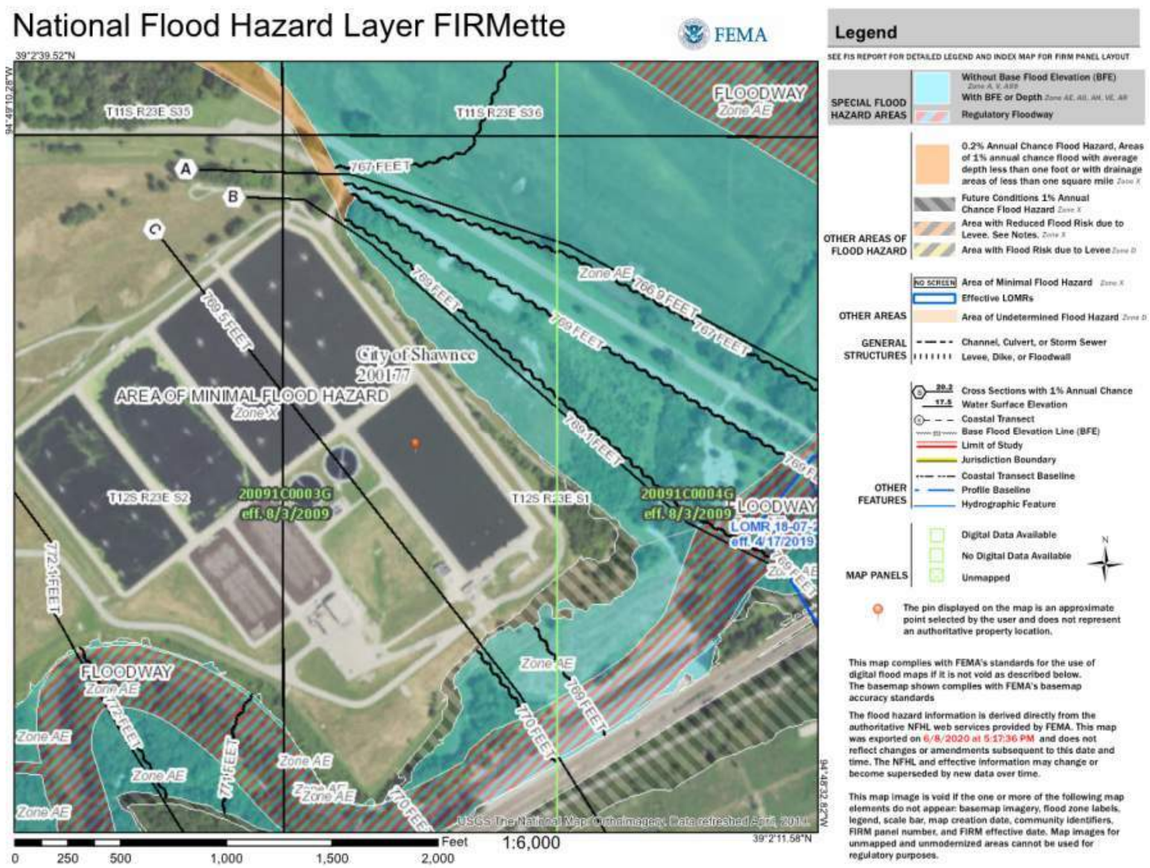


Figure 1-3 MCR FEMA Floodplain Map

2.0 Site Optimization

The MCR WWTP Expansion site layout needs to be optimized to allow for efficient plant operations and to minimize the cost of construction. Some considerations are as follows:

Operational Efficiency

- Provide all-weather access to site.
- Provide redundancy to critical processes and piping to eliminate single points of failure.
- Layout plant so that influent is only pumped once as it flows through the plant.
- Orient the facilities to provide convenient access for maintenance and removal/replacement of equipment.
- Provide convenient ingress/egress routes to facilities with frequent chemical deliveries or residuals removal – headworks, chemical feed systems, dewatered biosolids, etc.
- Provide looped road network to minimize the “dead-end” turnaround areas.
- Locate septage receiving such that drivers have convenient deliveries while also restricting access to the main plant site.
- Locate Administration Building to balance convenience of access for outside visitors and for staff.

Cost Minimization

- Reuse of facilities as it makes sense based on hydraulics and capacity.
- Orient the facilities to minimize the headloss through the process streams, thus minimizing the cost of pumping.
- Orient facilities to minimize lengths of large diameter pipe runs.
- Group facilities in close proximity that require common support functions, such as odor control.

In addition to optimizing the MCR layout for efficient plant operations and minimizing the costs of construction, the following sections describe key considerations used to optimize the layout.

2.1 SITE OPTIMIZATION FACTORS

2.1.1 Reuse of Existing Facilities

To minimize cost, it is preferable to reuse existing facilities to the extent possible. Reuse of facilities is dependent on a few factors such as site location, hydraulic capacity, and constructability. At the existing MCR WWTP, reuse of the final clarifiers, sludge pumping station, ultraviolet (UV) disinfection, influent pumping, and effluent tunnel were investigated.

The last plant expansion project at the MCR WWTP was Contract 6, which was completed in 2006. That project included construction of a mechanical plant with two final clarifiers (FCs), a sludge pumping station, and a UV disinfection facility. These facilities were designed to handle a peak flow of 24 million gallons per day (mgd); however, each of these facilities were constructed with the thought of future expansion. The clarifiers were located such that two additional identically sized clarifiers could be added in the future. The sludge pump station and UV building were constructed such that they could be added on to as part of a future expansion project. These facilities were designed such that future expansion could increase the secondary treatment peak capacity to 48 mgd.

As discussed in previous TMs, the proposed peak secondary flow for the MCR WWTP Expansion is 63 mgd. This means that if the existing clarifiers, sludge pump station, and UV disinfection facilities were to be reused, they would need to be expanded by approximately 62 percent instead of the originally designed 50 percent. The expansion, therefore, is not a simple duplication of the existing facilities. Below is a more detailed description of each facility.

2.1.1.1 Final Clarifiers and Sludge Pump Station

As discussed in TM 3 – Secondary and Sidestream Treatment, there are two options for the FCs at the MCR WWTP. Alternative 1 is to reuse the 2 existing clarifiers and build 3 more identically sized (130-ft diameter) units. Alternative 2 is to build 4 new units, each with a diameter of 145 feet, and then demolish the existing units.

These alternatives were considered in more detail as part of this TM. One key consideration in this evaluation is if the hydraulics would allow the existing clarifiers to fit in the new hydraulic profile. After a detailed review of the existing hydraulics and the hydraulics of the new facilities, it may be possible to reuse the two existing clarifier basins and also add three new clarifiers at the same elevation of the existing units; however, the hydraulics are very tight and do not provide enough margin at this stage of planning. To ensure that the clarifiers could be reused, the wall elevations of the existing units could be raised by three feet. This extension allows for more flexibility in the head loss based on potential future changes in site conditions. It should be noted that raising the wall elevations includes raising the v-notch weirs. This would make the clarifiers deeper, which may not be preferable.

Another key consideration in this evaluation is if there is a cost benefit of reusing the existing clarifiers. After considering all the required changes to reusing the final clarifiers and sludge pump station, reusing the existing facilities was estimated to be more costly than building new facilities. Reusing these facilities also complicates the construction sequence as it would require more connections to existing facilities and MOPO activities. For these reasons, constructing all new clarifiers and a sludge pump station will be used in site layouts at MCR.

2.1.1.2 UV Disinfection

As discussed in TM 5 – Disinfection Treatment, the existing disinfection system at the MCR WWTP is a Trojan Technologies, Inc. UV3000 Plus system. This system was a common technology in 2006 and is occasionally still installed today. However, UV disinfection is a rapidly changing technology, and the current state-of-the-art system is the TrojanUVSigna™ system. Although the existing MCR WWTP UV Disinfection building was designed for future expansion, the existing channels will require significant structural changes to be retrofitted to become compatible with a TrojanUVSigna™ system; therefore, as discussed in TM 5, it is recommended that a new UV Disinfection system be constructed and the existing be demolished.

2.1.1.3 Influent Pump Station and Effluent Tunnel

Most of the existing facilities at MCR will not be reused; however, two existing facilities that will remain in operation are the Influent Pump Station (IPS) and the effluent tunnel. These facilities are the start and the end of the treatment at the MCR WWTP. The IPS is in the southwest corner of the site. Most of this facility is original to the plant, including the coarse screens and wet weather pumps. The existing firm capacity of the dry weather pump station is approximately 24 mgd, while the existing firm capacity of the wet weather pump station is 39 mgd. The existing peak flow from the Mill Creek Interceptor is approximately 98 mgd, and the ultimate peak flow is approximately 116 mgd. It is therefore understood that MCR will need to increase the influent pumping capacity in the interim and for ultimate conditions as part of the MCR WWTP Expansion. It is likely that an additional pump station will be added to supplement the existing IPS; however, this analysis is discussed in TM 9 – Pumping. For the purposes of this TM, it is important to know that the location where the Mill Creek Interceptor arrives at the MCR WWTP will not be changed.

The gravity discharge effluent tunnel is a 96-inch HOBAS pipe that connects to the Kansas River effluent discharge pipe. The tunnel construction was completed in 2014. The existing diffuser was designed to discharge up to 105 mgd through 24-inch check valves; however, the check valves can be upsized in the future to 36-inch diameter to increase the flow to 132 mgd. For effluent to discharge MCR via the effluent tunnel, it must first enter the plant effluent junction box on the south side of the existing Lagoon Cell No. 8. The plant effluent junction box is where mechanical plant effluent is combined with lagoon effluent. Once combined, the plant effluent flows through a Parshall flume for metering, followed by the tunnel drop shaft. All existing infrastructure downstream of the plant effluent junction box will remain at the future MCR WWTP, meaning that the new facilities will tie into the plant effluent junction box.

2.1.2 Pumping Considerations

A preliminary estimate of the hydraulic profile for the MCR WWTP Expansion was developed by looking at the hydraulic profile of Tomahawk Creek (THC) WWTP. The annual average (AA) flow at the THC WWTP is 19 mgd and the treatment process is very similar, making it a good high-level comparison. Using the THC WWTP profile and a preliminary estimate of pipe losses to account for slightly more flow at the MCR WWTP, the preliminary hydraulic profile at the MCR WWTP has approximately 32 feet of headloss during peak secondary flow conditions.

The existing headloss through the mechanical plant at the MCR WWTP is 18 feet. Since the elevation of the effluent junction box is not changing, this means the influent pump station will need to pump to a higher static head condition by roughly 14 feet. As mentioned in Section 2.1.1.3, interim and ultimate improvements at the IPS will be required to meet the flow conditions. It is believed that these IPS improvements would install pumps that can meet this higher head condition. Based on this analysis, it is recommended to have gravity flow from headworks to the effluent tunnel. A more detailed summary of pumping at the MCR WWTP is discussed in TM 9 – Influent Pumping.

2.1.3 Unit Process Adjacency

When developing each of the various site plan alternatives, facilities were located on the site based upon several important factors. These factors primarily consisted of the following:

- Facilities with gravity flow between unit processes were given priority over those with pumped flow to manage overall headloss within the hydraulic profile.
- Facilities with large diameter pipe runs were given priority over those with smaller diameter pipe runs.
- The length of gravity sludge lines such as from primary clarification to primary sludge pumping was minimized.
- Solids processing facilities were located adjacent to each other when possible, to minimize odor control facilities.

2.2 LAYOUT CONSIDERATONS

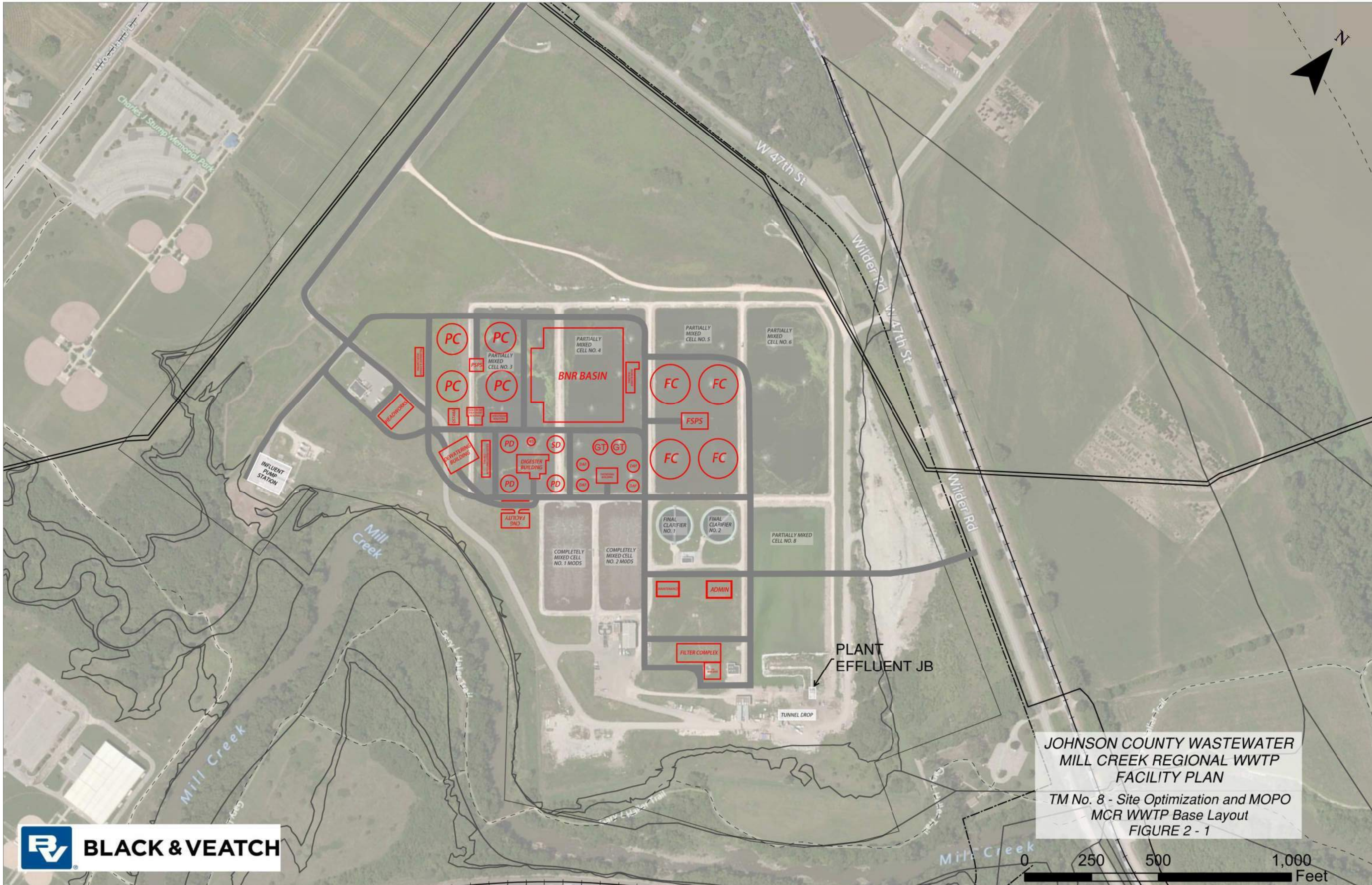
During the development of the preliminary site layouts for the MCR WWTP Expansion, the most fundamental consideration was the location of the future facilities. There were multiple options for constructing the facilities, including in the open space on the hillside in the northern corner of the site, in the footprint of the existing partially mixed lagoons, and in the footprint of the existing mechanical plant. Placing the new facilities in the footprint of the existing mechanical plant was immediately screened because the equipment must stay in service to maintain treatment during construction. Constructing the new facilities on the hillside would not be feasible because not only is the area highly visible, but it would require a large amount of sitework and likely even re-grading on Wilder Road. In addition, since flow comes into the site and leaves the site to the south, locating the facilities on the northern corner of site would not be optimal hydraulically. The final option, placing the new facilities in the footprint of the partially mixed lagoons, was determined to be the most advantageous.

Another key consideration in determining the final layout was the location of the solids processing facilities. Historically at MCR, winds from the east or north directions make the lagoons more noticeable at the neighboring recreational fields and nearby trail. Although the solids processing facilities will be odor controlled, the dewatering building will have trucks hauling off dewatered solids which have the potential for odorous escape and increasing the site visibility. The less visible the solids processing facilities are, the less potential for the surrounding areas to notice the daily operations of the MCR WWTP.

Preliminary locations for the solids processing facilities included the north side of Lagoon Cells 4 and 5, the south side of Lagoon Cells 4 and 5, Lagoon Cell 8, and within the footprint of the existing mechanical plant. It was decided that within the footprint of Lagoon Cells 4 and 5, the south side would be preferred because the facilities will be less visible from the surrounding roads. Locating the solids processing facilities in Lagoon Cell 8 or within the footprint of the existing mechanical plant could potentially reduce visibility and odors to the surrounding areas further; however, both locations will require construction phasing. More construction phasing and sequencing is estimated to add time and, as a result, cost to the construction schedule. Therefore, Cell 8 and within the footprint of the existing mechanical plant becomes less desirable. If solids processing facilities' visibility and potential odors become a greater concern in the interim, a phased construction approach could be further evaluated with input from a Contractor. The final layout will have the solids processing facilities located in the south part of Lagoon Cells 4 and 5, as shown in Figure 2-1.

2.3 FILTER COMPLEX AND UV DISINFECTION BUILDING LOCATION

The final key consideration discussed during the preliminary site layouts was the optimal location of the Filter Complex and UV Disinfection building. Three potential locations were evaluated, as discussed in the upcoming Sections. Evaluations were based on a planning-level comparison of large diameter pipe routing, connections to the Plant Effluent Junction Box, constructability, and cost.


JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN
TM No. 8 - Site Optimization and MOPO
MCR WWTP Base Layout
FIGURE 2 - 1
0
250
500
1,000
Feet
N
BLACK & VEATCH
W 47th St
Wilder Rd
Mill Creek
PC
PC
PC
PC
BNR BASIN
FC
FC
FC
FC
FSPS
PSPS
HEADWORKS
DEWATERING BUILDING
DIGESTER BUILDING
PD
PD
PD
SD
GT
GT
ADMIN
FILTER COMPLEX
INFLUENT PUMP STATION
PLANT EFFLUENT JB
TUNNEL DROP
PARTIALLY MIXED CELL NO. 3
PARTIALLY MIXED CELL NO. 4
PARTIALLY MIXED CELL NO. 5
PARTIALLY MIXED CELL NO. 6
PARTIALLY MIXED CELL NO. 8
FINAL CLARIFIER NO. 1
FINAL CLARIFIER NO. 2
COMPLETELY MIXED CELL NO. 1 MODS
COMPLETELY MIXED CELL NO. 2 MODS

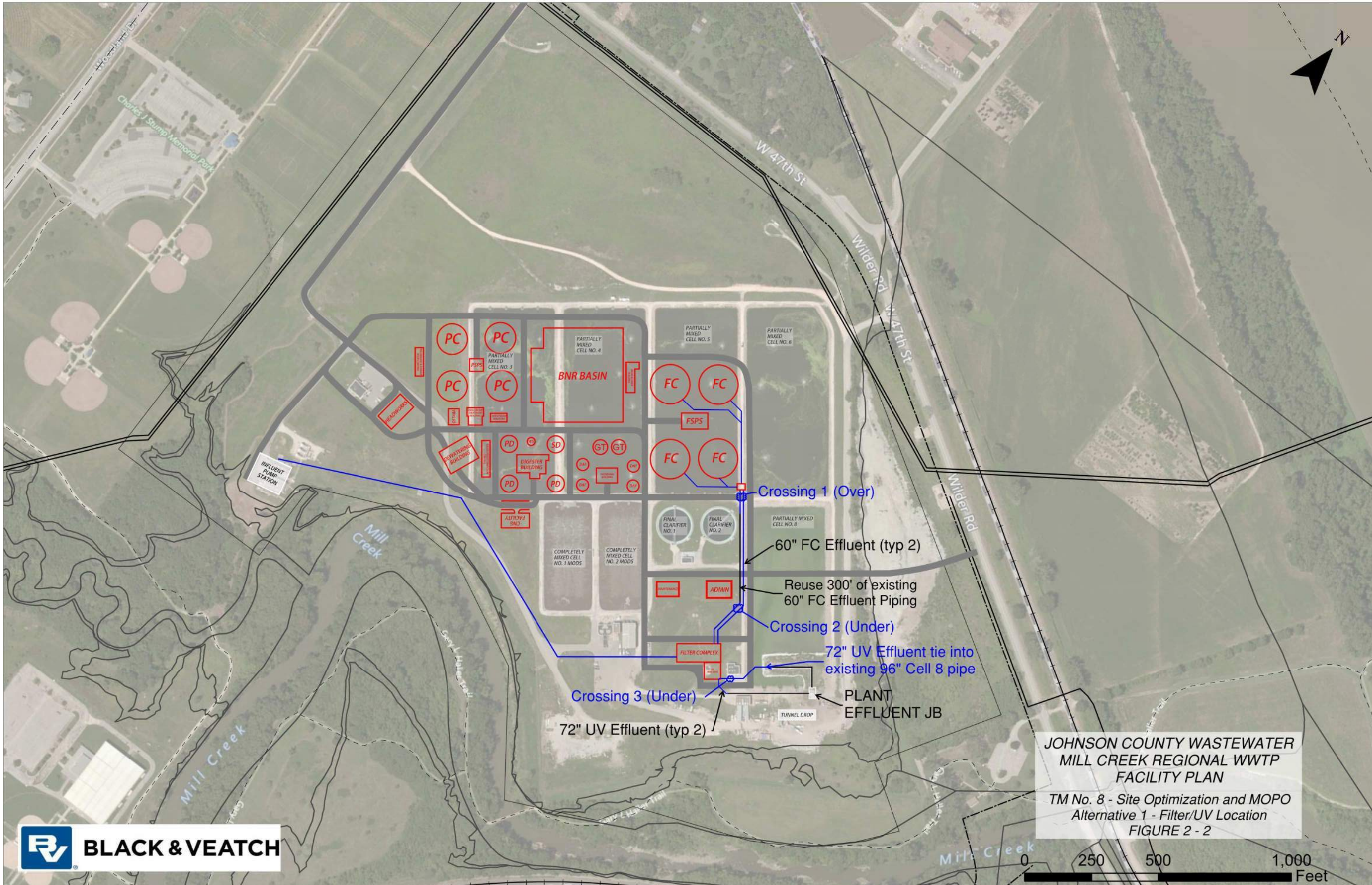
2.3.1 Alternative 1 - South in Former Cell No. 7

The Alternative 1 layout is based on locating the Filter Complex and UV Disinfection building on the south side of the site in the footprint of former Cell 7. This layout is presented in Figure 2-2.

There are several benefits of this layout. The biggest benefit is being able to start building these facilities from the first day of construction, without the need to pre-load Cell 7, resulting in the fastest completion. Once constructed, and once the new force main from the IPS is installed, the Filter Complex could be used to treat flows exceeding 24 mgd. Using the Filter Complex for wet weather treatment instead of Cell 8 provides more control to JCW and a more reliable level of treatment. Another benefit of this layout is it minimizes the length of the largest diameter piping. Since the Filter Complex and UV Disinfection facilities are sized to handle a maximum flow of 6Q, the UV Disinfection effluent piping is the largest piping onsite. It is estimated this pipe would be at least 72-inches — making it very expensive — so locating the Filter Complex and UV Disinfection facility as close as possible to the Plant Effluent Junction Box is the most cost-effective approach.

To increase the redundancy of the UV Disinfection effluent piping, there are two effluent pipes that connect to the Plant Effluent Junction Box. One pipe is routed east and eventually ties into the 96-inch Cell 8 effluent pipe. This pipe is 72-inches until it connects to the larger pipe. The 96-inch pipe has the morning glory weirs. Once those weirs and the dished bulkhead are removed and reconnected, the line can be put into service; however, by reusing the 96-inch pipe that was installed as part of the Contract 10 work at the MCR WWTP, there is a cost savings compared to installing all new piping. The second pipe is also 72 inches and is routed to the south. This pipe ties into the existing 72-inch mechanical plant effluent line. Both lines eventually tie into the Plant Effluent Junction Box.

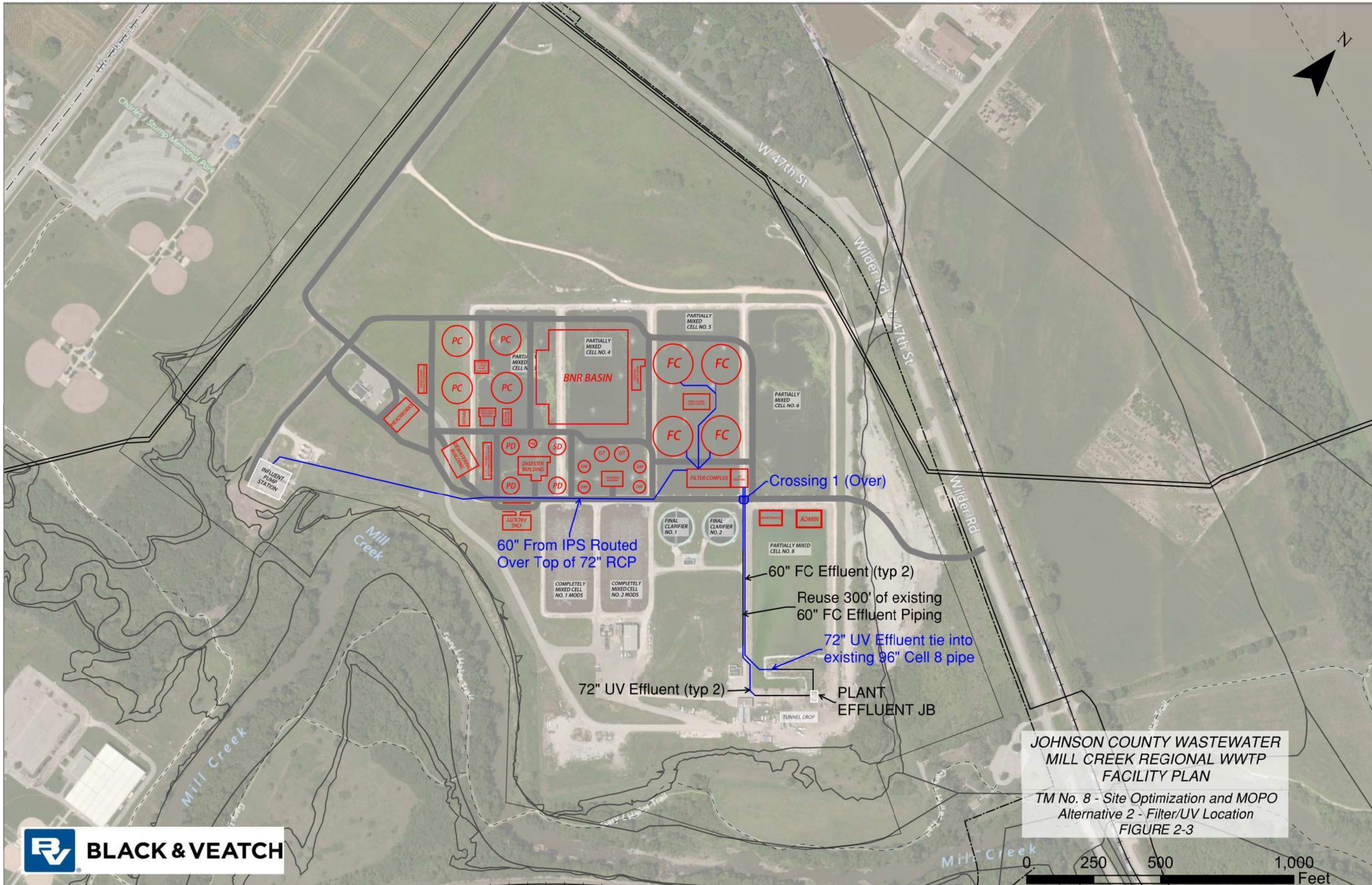
The biggest disadvantage of Alternative 1 is the multiple crossings of in-service piping, as shown in Figure 2-2. There are three different areas where new piping will need to cross large-diameter in-service piping. The first crossing is just south of the Final Clarifier Splitter Box. To connect the Filter Complex to the Final Clarifier effluent, crossing the 72-inch RCP wet weather header to Cell 8 is required. The top of this existing pipe is approximately 10 feet below grade, so routing the new pipe over top can be done. The second pipe crossing is also on the run of pipe connecting the FCs and the Filter Complex. The existing final clarifier effluent piping is routed in this corridor and connects to the existing UV building. Since the Filter Complex is west of the existing UV building, at some point the new Final Clarifier effluent pipe must cross the existing final clarifier effluent pipe. The top of the existing pipe is approximately 5 feet below grade, so it is likely that the new pipe will have to go underneath. Crossing underneath an in-service pipeline could require temporary bracing of the existing pipe, and there is always the possibility of damaging the existing pipe. The final pipe crossing is just south of the existing UV building. The new 72-inch UV effluent pipe heading to the east crosses the existing 60-inch UV effluent pipe. The existing pipe is about 5-feet below grade, so the new pipe will need to be routed underneath. Overall, the piping connections increase the complexity, but it could be completed with additional construction measures to protect the existing piping during installation.


JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN
TM No. 8 - Site Optimization and MOPO
Alternative 1 - Filter/UV Location
FIGURE 2 - 2
N
W 47th St
Wilder Rd
Charles J Stump Memorial Park
Mill Creek
INFLUENT PUMP STATION
HEADWORKS
PC
PSPS
BNR BASIN
PARTIALLY MIXED CELL NO. 3
PARTIALLY MIXED CELL NO. 4
PARTIALLY MIXED CELL NO. 5
PARTIALLY MIXED CELL NO. 6
PARTIALLY MIXED CELL NO. 8
FC
FSPS
DEWATERING BUILDING
PD
SD
DIGESTER BUILDING
ST
FINAL CLARIFIER NO. 1
FINAL CLARIFIER NO. 2
COMPLETELY MIXED CELL NO. 1 MODS
COMPLETELY MIXED CELL NO. 2 MODS
ADMIN
FILTER COMPLEX
TUNNEL DROP
Crossing 1 (Over)
60" FC Effluent (typ 2)
Reuse 300' of existing
60" FC Effluent Piping
Crossing 2 (Under)
72" UV Effluent tie into
existing 96" Cell 8 pipe
Crossing 3 (Under)
PLANT
EFFLUENT JB
72" UV Effluent (typ 2)
BLACK & VEATCH
0
250
500
1,000
Feet

2.3.2 Alternative 2 - South Side of Cell No. 5

The layout for Alternative 2 is shown in Figure 2-3. This alternative is based on the Filter Complex and UV Disinfection facility located in the south part of lagoon Cell 5. As seen on the layout, this alternative results in the most compact final site arrangement. Additional benefits associated with this layout include minimizing the length of piping from the FCs to the Filter Complex, minimizing the length of piping from the IPS to the Filter Complex, and reducing the number of in-service pipe crossings to one.

There are a few disadvantages with this site layout. The first is that the Filter Complex and UV disinfection facility can't be constructed until Cell 5 has been pre-loaded to minimize settlement. It is believed that this process could take up to a year, so this would increase the duration of using Cell 8 for wet weather treatment by that same period of time. The second disadvantage is this layout has much longer UV effluent piping. Although there is a cost savings by having less piping from the IPS and FCs to the Filter Complex, the net result is an increase in piping costs since the UV effluent piping is the largest diameter piping onsite. The final potential challenge with this layout is installing the piping from the IPS to the Filter Complex. This piping is basically a parallel route over the 72-inch RCP to Cell 8 that is fairly deep, so there is sufficient depth to install this piping; however, the complexity of installing piping through a berm adjacent to active lagoons should not be understated. There is the possibility that sheet piling and dewatering would be required to keep this area dry during the installation. This would add significant costs to this layout alternative.



W 47th St
Wilder Rd
Charles J Stump Memorial Park
Mill Creek
PC
FC
BNR BASIN
PARTIALLY MIXED CELL NO. 4
PARTIALLY MIXED CELL NO. 5
PARTIALLY MIXED CELL NO. 6
PARTIALLY MIXED CELL NO. 8
HEADWORKS
INFLUENT PUMP STATION
PD
SD
DIGESTER BUILDING
GT
FILTER COMPLEX
ADMIN
FINAL CLARIFIER NO. 1
FINAL CLARIFIER NO. 2
COMPLETELY MIXED CELL NO. 1 MODS
COMPLETELY MIXED CELL NO. 2 MODS
TUNNEL DROP
Crossing 1 (Over)
60" From IPS Routed Over Top of 72" RCP
60" FC Effluent (typ 2)
Reuse 300' of existing 60" FC Effluent Piping
72" UV Effluent tie into existing 96" Cell 8 pipe
72" UV Effluent (typ 2)
PLANT EFFLUENT JB
JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN
TM No. 8 - Site Optimization and MOPO
Alternative 2 - Filter/UV Location
FIGURE 2-3
0
250
500
1,000
Feet
N
BLACK & VEATCH

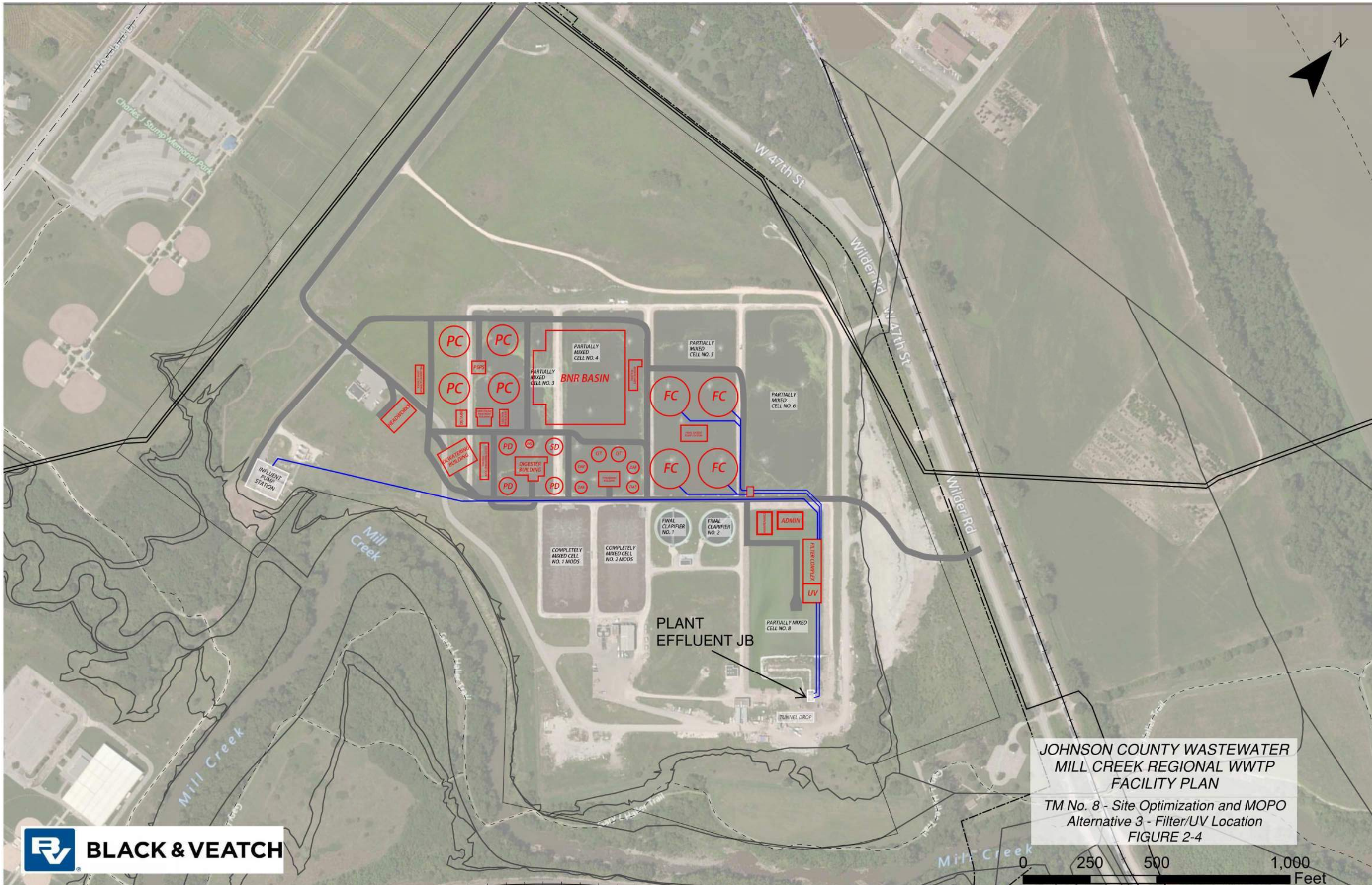
2.3.3 Alternative 3 - North Side of Cell No. 8

The layout for Alternative 3 is shown in Figure 2-4. This alternative is based on the Filter Complex and UV Disinfection facility located in Cell 8. The primary goal of this layout was to be a blend of Alternatives 1 and 2. This layout combines a somewhat compact final site arrangement while also trying to reduce the length of UV effluent piping. Another benefit of this layout is it has no in-service pipe crossings.

Although this alternative was an attempt at a hybrid between Alternatives 1 and 2, it has the most disadvantages. Most of the issues with this layout come from complications of building in Cell 8. To build structures in Cell 8 and install piping to those structures, sheet piles will need to be installed to effectively isolate the eastern part of the cell. This isolated area will need to be dewatered, and it is likely that the piping will need to be supported using cradles and piles. This process of installing sheet piles, dewatering, backfilling, compacting, and installing pipe supports will add length to the construction schedule, all before any construction on the structures begins. In addition, by using part of Cell 8, there is less volume for the wet weather treatment and the TSS storage. It is likely that this will require an increased frequency of cleaning. The layout also has the longest length of piping from the IPS to the Filter Complex, and the installation of that piping is subject to the same concerns discussed in Alternative 2.

2.3.4 Recommendation

The recommended location of the Filter Complex and UV Disinfection facility, based on the planning level evaluation the MCR WWTP Expansion, is as shown in Alternative 1, Figure 2-2. This layout provides the greatest benefits from a construction and cost perspective.



JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN

TM No. 8 - Site Optimization and MOPO
Alternative 3 - Filter/UV Location
FIGURE 2-4

BLACK & VEATCH

2.4 SITE CIVIL CONSIDERATIONS

There are several additional miscellaneous site constraints that should be considered. Some of these considerations are discussed in greater detail in other TMs; however, since they show up on the final site plan, a brief discussion is included in this TM. The site civil work associated with the MCR WWTP Expansion includes excavation and fill, roadways, septage receiving and jet-vac, stormwater management, and effluent tunnel flushing. These considerations have been incorporated into the recommended site layout for the MCR WWTP Expansion.

2.4.1 Site Excavation/Fill

A key site civil consideration is excavation and fill. It is understood that the existing lagoon cells will be going away as part of the MCR WWTP Expansion. Lagoon Cells 3, 4, 5, 6, and 8 each have a maximum liquid volume of approximately 82,000 cubic yards (cy). Lagoon Cells 1 and 2 both have a maximum liquid volume of 31,500 cy. The total existing liquid storage in the MCR lagoon cells is nearly 480,000 cy. If this storage volume of dirt was placed as fill on a football field, the depth would be 225 feet. In other words, a lot of fill is going to be required to bring the lagoon cells up to the top of the berm elevation. Each MCR WWTP Expansion structure that is located within the existing lagoon footprint will reduce the amount of fill that is needed, which will also reduce the sitework costs.

The upcoming Section 3 of this TM discusses the modifications that are required to maintain treatment during construction of the MCR WWTP Expansion. As explained in that Section, lagoon Cells 1, 2, 6, and 8 are needed to maintain treatment. Cells 6 and 8 must be converted to handle solids and wet weather treatment before the other partially mixed lagoons are removed from service. That leaves Cells 3, 4, and 5 for the bulk of the future structures. Based on previous subsurface investigations at MCR, it is anticipated that prior to construction in any of the existing lagoon cells, each cell will need to be pre-loaded, similar to former Lagoon Cell 7. The pre-loading process consists of installing wick drains that terminate in a granular drainage layer and then adding fill on top to promote quicker settlement of the filled area prior to construction. When former Lagoon Cell 7 was pre-loaded, fill was added until the depth above the bottom of the lagoon was roughly 20 feet. An allowable bearing capacity of 2,000 pounds per square foot (lb/sf) was used for the Cell 7 pre-loading. It is estimated that the bearing capacity for these additional cells would be similar. Overall, the Cell 7 fill and settlement process took about seven months. Time for pre-loading Lagoon Cells 3, 4, and 5 is built into the preliminary schedule that is presented in TM 10 - Implementation.

The expected sitework workflow for the MCR WWTP Expansion starts off with installing the piping necessary to get solids to Cell 6 and all wet-weather flow to Cell 8. Once the piping is installed, filling and preloading Cells 3, 4, and 5 can begin. Once sufficient settlement has occurred, post-settlement survey will be conducted to confirm that the final ground surface elevation is at the appropriate level and then the area can be excavated to build the MCR WWTP Expansion structures. Once the new structures are completed and placed into service, the existing mechanical plant and Lagoon Cells 1, 2, 6, and 8 can be decommissioned. The remaining lagoon cells will be graded as needed, and old structures will be demolished. The sitework workflow summary is listed below in three phases:

- Phase 1 – Offsite Fill/Pre-load of Lagoon Cells 3, 4, and 5
- Phase 2 – Construction and Start-up of MCR WWTP Expansion Facilities
- Phase 3 – Demolition of Existing MCR WWTP Facilities and Final Site Grading

Phase 1 includes filling and pre-loading Lagoon Cells 3, 4, and 5. Previously, it was mentioned that each of these cells has 82,000 cy of volume; however, it is expected that, similar to Cell 7, the pre-loading will require these cells to be filled to an elevation higher than the existing high-water level, which increases the total amount of volume for each cell. In addition, any time fill is added, a consolidation factor needs to be applied. The consolidation factor corrects the amount of fill by estimating how much compaction will be achieved. After increasing the fill elevation and applying a consolidation factor of 20 percent, Lagoon Cells 3, 4, and 5 will each require 160,000 cy. It is believed that this amount of soil will have to be brought in from offsite due to the large volume. It is estimated that this process will take up to one year to complete.

Phase 2 includes excavating the site as needed to build the new structures. The Contractor would be able to move soil around the site as needed, as long as treatment through the mechanical plant and use of Cells 6 and 8 is maintained. Once structures are completed, the Contractor would be able to backfill around each structure in accordance with the overall site grading.

In Phase 3, all new facilities would be in operation, thus making the existing mechanical plant and Cells 6 and 8 no longer necessary. Phase 3 includes the demolition of these facilities, and the excavation and backfill associated with that process. Existing Cells 1, 2, 6, and 8 will need to be filled, but not to the depth of the previous lagoon cells. The overall site grade at the MCR WWTP slopes to the south and west of the site matching the natural topography, so it is acceptable if these lagoons are not at the same grade as the center of the site. No additional site pre-loading is expected in these areas due to the lack of structures in these former lagoon cells. Minor settlement is not an issue when there is not a building on the area that is settling as long as ponding does not occur. The final component of Phase 3 is the final site grading. The Contractor will move soil around as needed to promote site drainage and to match the existing topography as much as possible. Overall, the site will be relatively flat, with the highest site grade near existing lagoon Cell 3 and sloping away towards the surrounding Mill Creek and Kansas River.

2.4.2 Site Access and Plant Roads

As part of this project, several access points to the site were analyzed. The existing facility has two access points. The primary access is on the north end of the site off West 47th Street. This access road is used by plant staff and all visitors to the site. The difficulty of having the primary access road here is that it is on top of a hill. The hill and surrounding topography limits visibility when making the entrance or exit turn. This turn can be especially difficult for large vehicles, such as trucks making deliveries or hauling. The secondary access is on the northeast, just west of existing Lagoon Cell No. 6. This entrance is rarely used and is below the 500-year flood elevation. Although the topography is flatter in this area, the entrance and exit turns are still relatively blind due to the turn on West 47th Street.

After looking at the existing site topography and discussing with JCW, it was decided to add an access road off Wilder Road, to the east of existing Lagoon Cell 8. Locating an access road in this area has many benefits, the most of which is improved visibility for drivers turning off Wilder Road. It is recommended that this road becomes the primary plant access due to the improved visibility. One downside is the elevation of Wilder Road east of the MCR WWTP, which is below the 100-year floodplain elevation. Because of this, the access road will have a slope of approximately 5 percent as vehicles approach the site. Additionally, this plant access will not be available during flood conditions. In these situations, the current primary plant access road will be used instead. The current primary plant access road will be maintained to provide secondary access during dry weather and primary access during wet weather. All MCR WWTP layout alternatives include these two ways of site access.

It should be noted that any time roadway improvements are recommended, they will need to be approved by the City of Shawnee. Conformance with KDOT or AASHTO requirements for safe sight distance will likely be required as part of any changes to the entrance. Additional roadway design that accounts for heavy truck traffic, including the addition of a left-turn lane, would be recommended to protect Wilder Road.

Once onsite, the MCR WWTP facility will have a looped road network minimizing the number of dead ends. All facilities will have adequate access roads for operation and maintenance purposes. The plant roads' widths are set at 25 feet from back of curb to back of curb.

2.4.3 Septage Receiving and Jet Vac

Both septage receiving and Jet Vac are discussed in detail in TM 7 – Support Facilities. While neither of these facilities are drivers of the site layout, they should be discussed because they will show up on the final layout.

JCW accepts septage waste to the MCR WWTP from approximately 60 haulers per week. These haulers are not affiliated with JCW, so the design of the septage receiving system is focused on limiting site access. The final site plan will include a loop where haulers can pull onto the site, be able to drive the loop, and complete their delivery without driving across the main treatment facilities. During discussions with JCW, there has been mention of the possibility of improving the existing septage receiving facilities prior to the MCR WWTP Expansion. If the primary septage receiving location does not facilitate improvement prior to the MCR WWTP Expansion, a secondary option will be shown on the final site layout.

The future Jet Vac dumping station is only for JCW trucks, so limiting site access is not needed. The most important features are locating it in a spot that is easy for drivers to get to and its proximity to the Headworks building. These considerations will be incorporated into the final Jet Vac location.

2.4.4 On-Site Stormwater Detention Basin

The MCR WWTP Expansion will decrease the amount of permeable soil at the MCR WWTP. As such, it is important to develop a solution to handle the on-site stormwater drainage. The stormwater detention basin was sized based on the United States Department of Agriculture (USDA) Soil Conservation Service (SCS) Technical Release 55 (TR-55) method, the American Public Works Association (APWA) Section 5600, and the City of Shawnee Standards. The basin has a depth of approximately 8 feet, and a side length of 250 feet. An ideal location is one that fits in with the natural site topography and an area that could make practical use of former lagoons. As such, the square basin has been located within the footprint of Cells 1 and 2 as shown on the recommended site layout.

2.4.5 Effluent Tunnel Flushing

As part of the MCR WWTP Contract 10 work, where the 96-inch diameter tunneled pipeline was constructed, an operations and maintenance (O&M) plan was prepared. The O&M plan describes the operational, inspection, and maintenance requirements for the effluent tunnel, including the inlet and outlet components associated with the effluent tunnel. One of the potential maintenance concerns with the effluent tunnel was the accumulation of sediment over long periods of low flow conditions (less than 18 mgd). The recommended approach to address the sediment build-up is to flush the tunnel with the liquid in Cell 8, once a quarter to scour out the accumulated sediment. This "tunnel flush" procedure is outlined as an appendix to the O&M plan. Cell 8 is used to flush the tunnel because it is the last cell in the lagoon train and holds enough liquid that can be rapidly discharged. When developing potential layouts for the MCR WWTP Expansion, the "tunnel flushing" feature was reviewed to confirm that flushing of the effluent tunnel will not be negatively impacted by removing Cell 8 from the final site layout.

From the MCR WWTP Effluent Tunnel O&M Plan, Appendix A, the average dry weather flow upon commissioning of the effluent tunnel was expected to be 12 mgd. Through the 96-inch tunnel, this corresponds to a flow velocity of 0.37 feet per second (FPS). At this velocity, there is virtually no sediment transport capacity. It is estimated that this flow rate will deposit approximately 50 cf of sediment daily. Assuming constant sediment deposition, at this rate it would take over 700 consecutive days for the maximum sediment level of 12 inches to be reached. The MCR WWTP routinely sees flows exceeding 30 mgd or higher in wet weather events, and although the sediment load is higher for wet weather, the sediment transport capacity is much higher. A sustained 30 mgd flow over a 24-hour period has the potential to remove over 27,000 cf of sediment, which is over a year of daily deposition from a 12 mgd flow.

Based on the sediment removal capacity of wet weather events at MCR, it was decided that the removal of Cell 8 and the ability to flush the effluent tunnel is acceptable. Given that the ultimate projected daily average flow at MCR is 21 mgd, and the average frequency of wet weather events exceeding 2Q is between 14 days per year (presented in Table 3-4), this confirms that tunnel flushing after the MCR WWTP Expansion will occur naturally without the need for routine maintenance.

2.5 SITE PIPING

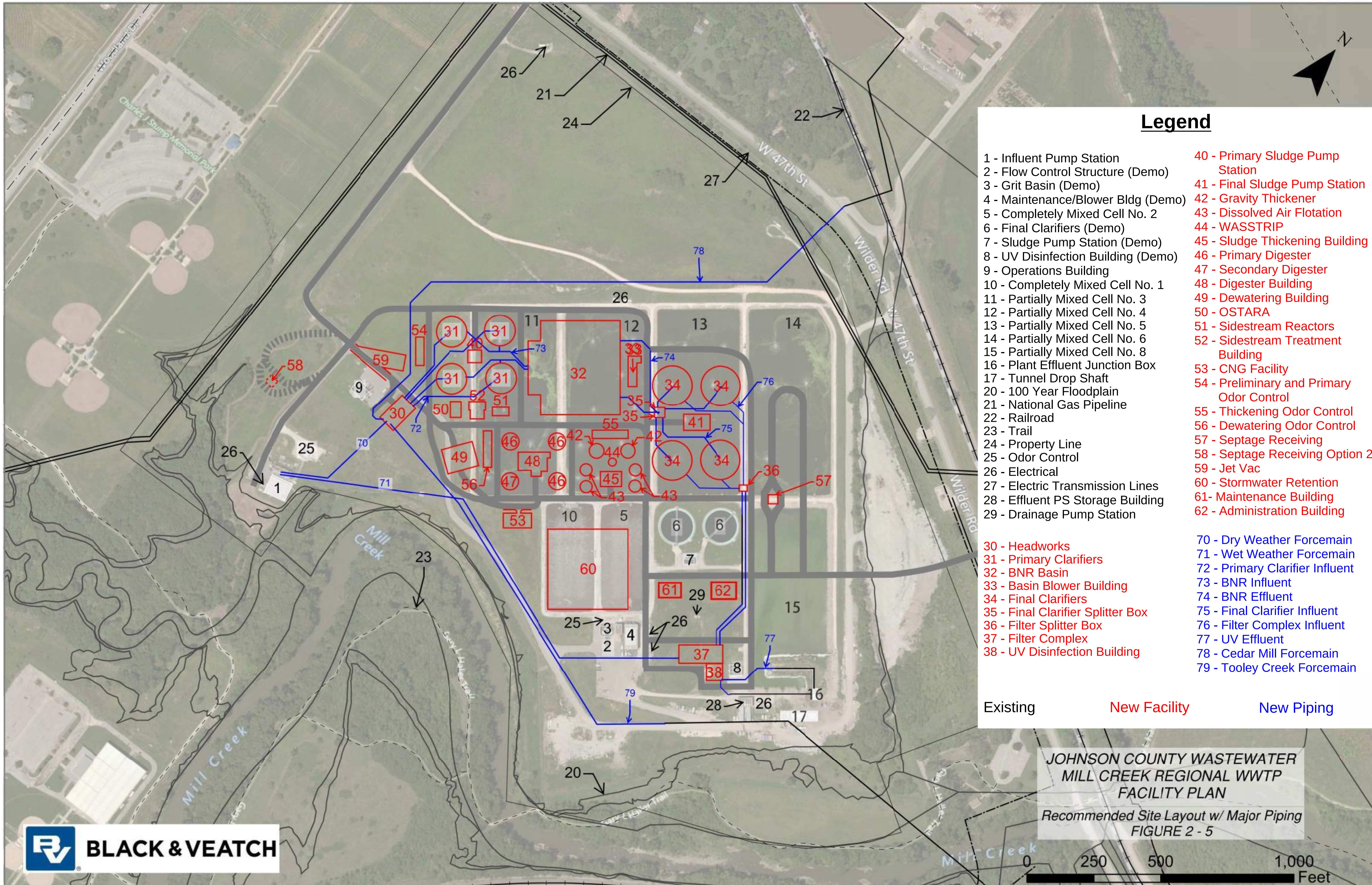
Based on the recommended site layout discussed in Section 2.3, site piping for major piping systems is described in this section. Figure 2-5 shows the recommended site layout along with a layout of all major piping between facilities. Table 2-1 shows the preliminary pipe sizing and corresponding velocities between each unit process. For gravity flow pipes, velocities less than 5 FPS are preferred to minimize the associated headloss. This, however, can result in the selection of larger diameter pipe that does not provide adequate velocities to prevent the settlement of solids within the pipe at low flows. To address this, dual pipes should be considered during detailed design for applicable pipe runs to maintain sufficient velocities at low flows while also trying to balance the maximum headloss between unit processes. Another important consideration associated with site piping is with BNR secondary treatment. It is important to reduce head fluctuations over the complete range of flows to minimize the fall over weirs, which entrains air and consumes carbon for the BNR process.

It should also be noted that piping shown in Table 2-1 and Figure 2-5 is only liquid process piping. It is expected that sludge piping from the FCs to the Final Sludge Pump Station, and from the Final Sludge Pump Station to BNR, is expected to be larger than 12 inches. This piping is not currently shown; however, it will be sized to minimize settlement of solids at all flow ranges.

Table 2-1 Summary of Major Plant Piping

SERVICE	SIZE (IN)	AA FLOW (MGD)	AA VELOCITY (FPS)	MAX FLOW (MGD)	MAX VELOCITY (FPS)
Tooley Creek Force Main (FM)	16	2	2.20	6.5	7.20
Cedar Mill Force Main (FM)	16	1	1.11	3.5	3.88
IPS to Headworks (FM)	60	21	1.65	63	4.96
IPS to Filter Complex (FM)	60	-	-	63	4.96
Headworks to PCs[1]	36	5.25	1.15	21	4.60
PCs to BNR[1]	48	10.5	1.29	42	5.17
BNR to FC Splitter Box	42	10.5	1.69	31.5	5.07
Splitter Box to FCs[1]	36	5.25	1.15	21	4.60
FCs to Filter Complex	42	10.5	1.69	31.5	5.07
UV to Plant Effluent JB[2]	72	21	1.15	63	3.45

- Splitter box to FCs maximum conditions are based on one unit out of service
- UV to Plant Effluent JB based on dual pipes from UV to Plant Effluent JB

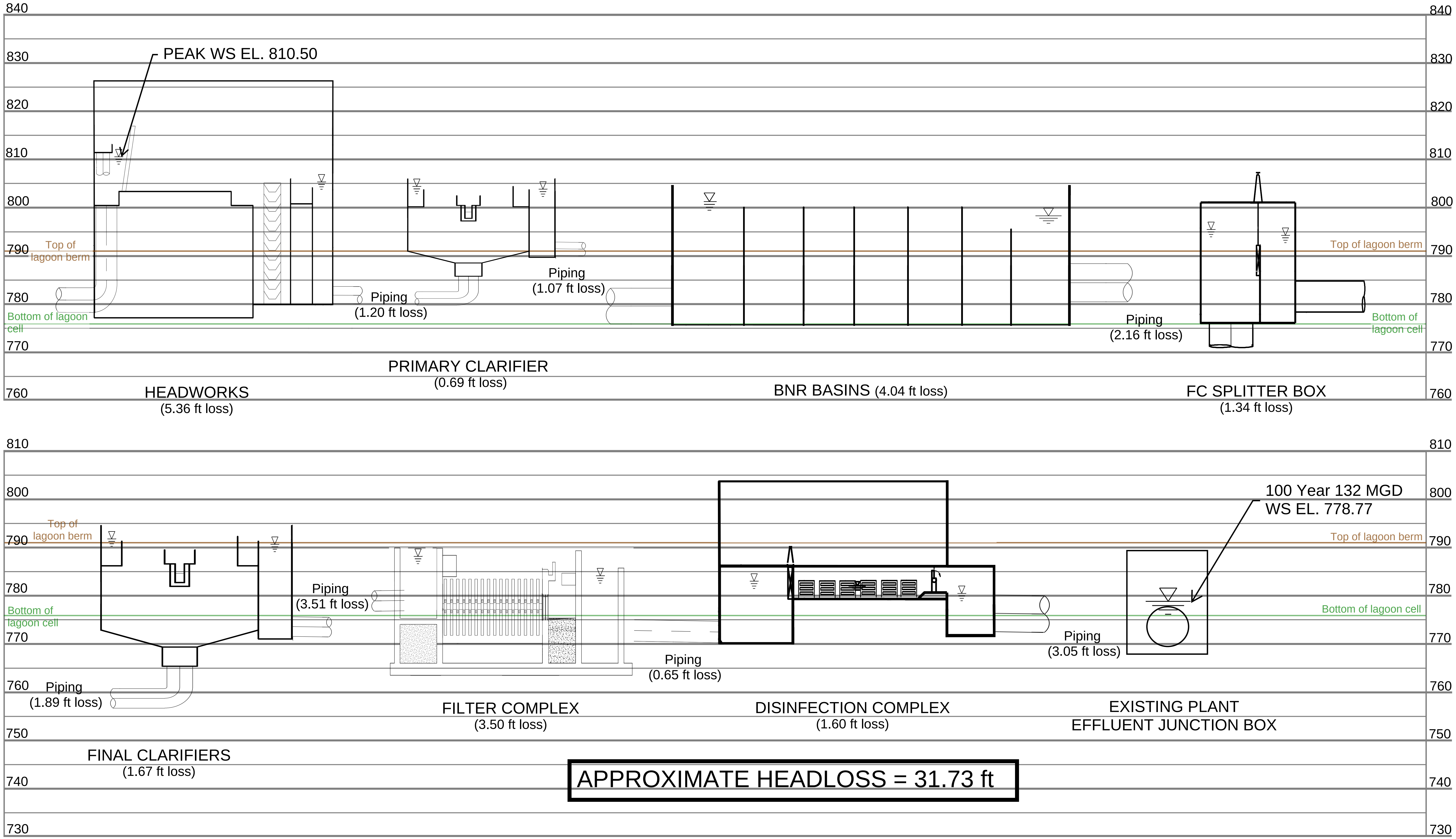

Legend
1 - Influent Pump Station
2 - Flow Control Structure (Demo)
3 - Grit Basin (Demo)
4 - Maintenance/Blower Bldg (Demo)
5 - Completely Mixed Cell No. 2
6 - Final Clarifiers (Demo)
7 - Sludge Pump Station (Demo)
8 - UV Disinfection Building (Demo)
9 - Operations Building
10 - Completely Mixed Cell No. 1
11 - Partially Mixed Cell No. 3
12 - Partially Mixed Cell No. 4
13 - Partially Mixed Cell No. 5
14 - Partially Mixed Cell No. 6
15 - Partially Mixed Cell No. 8
16 - Plant Effluent Junction Box
17 - Tunnel Drop Shaft
20 - 100 Year Floodplain
21 - National Gas Pipeline
22 - Railroad
23 - Trail
24 - Property Line
25 - Odor Control
26 - Electrical
27 - Electric Transmission Lines
28 - Effluent PS Storage Building
29 - Drainage Pump Station
30 - Headworks
31 - Primary Clarifiers
32 - BNR Basin
33 - Basin Blower Building
34 - Final Clarifiers
35 - Final Clarifier Splitter Box
36 - Filter Splitter Box
37 - Filter Complex
38 - UV Disinfection Building
40 - Primary Sludge Pump Station
41 - Final Sludge Pump Station
42 - Gravity Thickener
43 - Dissolved Air Flotation
44 - WASSTRIP
45 - Sludge Thickening Building
46 - Primary Digester
47 - Secondary Digester
48 - Digester Building
49 - Dewatering Building
50 - OSTARA
51 - Sidestream Reactors
52 - Sidestream Treatment Building
53 - CNG Facility
54 - Preliminary and Primary Odor Control
55 - Thickening Odor Control
56 - Dewatering Odor Control
57 - Septage Receiving
58 - Septage Receiving Option 2
59 - Jet Vac
60 - Stormwater Retention
61- Maintenance Building
62 - Administration Building
70 - Dry Weather Forcemain
71 - Wet Weather Forcemain
72 - Primary Clarifier Influent
73 - BNR Influent
74 - BNR Effluent
75 - Final Clarifier Influent
76 - Filter Complex Influent
77 - UV Effluent
78 - Cedar Mill Forcemain
79 - Tooley Creek Forcemain
Existing
New Facility
New Piping
W 47th St
Wilder Rd
Mill Creek
JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN
Recommended Site Layout w/ Major Piping
FIGURE 2 - 5
0 250 500 1,000
Feet
BLACK & VEATCH

2.6 HYDRAULIC PROFILE

Figure 2-6 presents the hydraulic profile based on the recommended MCR WWTP Expansion layout. The figure depicts approximate top-of-concrete elevations for each unit process and how these respective elevations relate to the top and bottom of the existing lagoon cells. Most future facilities will require excavation below the bottom of the existing lagoons. The figure also depicts preliminary hydraulic elevations.

The hydraulic profile was prepared using assumed headloss through each facility based on the modeled headloss for each facility at THC WWTP, with pipe losses connecting the facilities based on the final MCR layout. The THC WWTP has very similar unit processes and similar peak flows. For unit processes that were not at THC but are at MCR, such as the UV disinfection, the actual hydraulics were modeled based on preliminary layout drawings. The piping losses between each of the facilities is based on the recommended site layout shown. Based on these methods, the approximate headloss through all facilities is 32 feet.

When flows at MCR are 132 mgd and the Kansas River is at a 100-year flood condition, flows would back up to an elevation of 778.76 ft in the plant effluent junction box according to the effluent tunnel project preliminary design report. This is the assumed worst-case condition at MCR for the plant effluent junction box. Using this elevation as a starting point, and the hydraulic losses previously discussed, the high-water surface elevation in the Headworks Building would be 810.50. This is an increase of approximately 14 feet from the existing high-water surface elevation at MCR. Since improvements already need to be made to the IPS, it is preferred to increase the influent hydraulic capacity rather than add intermediate pumping on the site. This “pump once” profile (gravity flow from headworks to the effluent tunnel) was discussed with JCW during a biweekly meeting, and it was agreed that avoiding intermediate pumping is the desired alternative.



100 Year Flood Elevation = 773 ft

500 Year Flood Elevation = 779 ft

BLACK & VEATCH

JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN
TM No. 8 - SITE OPTIMIZATION AND MOPO
HYDRAULIC PROFILE
FIGURE 2-6

3.0 MOPO Considerations

With a recommended site layout, a more complete understanding of the required MOPO considerations can be developed. From TM 1 – Background, Flows, Loadings, and NPDES Limits; historical MCR influent flow data from the IPS meter vault shows that the current average daily flow at MCR is 10.5 mgd. The MCR Plant Expansion is expected to be completed by 2035, and the estimated daily average flow at that time is expected to be 12.0 mgd. Treatment of the daily average and wet weather flows to meet permit limits is required throughout construction. In addition, it is equally as important to develop a plan to handle the solids produced at MCR throughout construction.

3.1 DRY WEATHER TREATMENT STRATEGY

Table 3-1 presents a summary of the projected MCR WWTP Expansion start up conditions and the associated existing facilities needed to meet existing treatment levels. This table does not account for diurnal low flows at startup conditions. While it is important to understand the impact of these flows, specifically on influent pumping and grit removal, this section is focused on maintaining the existing treatment during construction. Therefore, since the existing diurnal low flow conditions are handled by the existing plant facilities, they will continue to be handled during construction.

Table 3-1 Dry Weather Treatment Summary

	CONDITION	INFLUENT FLOW (MGD)	CELLS IN SERVICE IN MECHANICAL PLANT	MLSS (MG/L)[1]	RAS FLOW (MGD)	CLARIFIERS IN SERVICE	SLR (LB/SF/D)	SOR (GPD/SF)
Current	AA Maint.	10.5	1	2,500	10.5	1	33	400
	AA	10.5	1	2,500	10.5	2	16	400
	MM	15.8	1	2,800	14	2	26	600
Projected Start-Up	AA Cell Maint.	12	1	2,900	12	2	22	450
	AA Clarifier Maint.	12	1&2	1,600	12	1	24	450
	AA	12	1&2	1,600	12	2	12	450
	MM	18	1&2	2,400	14	2	26	680
	PD	24	1&2	2,400	14	2	29	900

- MLSS determined at 10°C and 10-day aerobic SRT

Table 3-1 indicates that MLSS, SLR, and SOR at peak day conditions are within acceptable ranges and are similar to current MCR operation. In addition, the AA Maintenance condition is when one unit is out of service shows that conditions are also within acceptable ranges. Below, Table 3-2 presents a summary of the installed aeration blowers, and Table 3-3 presents projected startup airflow requirements at the MCR WWTP.

Table 3-2 Installed Aeration Blower Summary

TYPE	NUMBER OF UNITS	FLOW RATE PER UNIT (SCFM)	TOTAL FIRM CAPACITY (SCFM)	DISCHARGE PRESSURE	MOTOR, EA (HP)
Single Stage, Centrifugal	3	18,000	36,000	9.5	1,000

Table 3-3 Projected Startup Summary

	CONDITION	TEMP (°C)	INFLUENT FLOW (MGD)	CELL IN SERVICE	REQUIRED AIRFLOW (SCFM)
Projected Start-Up	MM	23	18	1&2	26,000
	PD	23	24	1&2	32,000

Table 3-3 confirms that the installed blower capacity is adequate to meet future peak day start-up required airflow. Similar to the low flow effect on pumping and grit removal, the existing minimum air conditions at the MCR WWTP are not changed and are therefore not discussed in Table 3-3. Based on information in Table 3-1, Table 3-2, and Table 3-3, it is recommended that existing Cells 1 and 2, the single stage centrifugal blowers, FCs 1 and 2, and the existing UV disinfection facility remain online throughout construction to maintain current treatment levels up to 24 mgd.

3.2 WET WEATHER TREATMENT STRATEGY

The recommended site layout shows most new facilities within the footprint of the existing lagoon cells. Since flow comes into the site and leaves the site to the south, locating new facilities further south minimizes the length of site piping, and therefore minimizes headloss. As part of the planning process, a wet weather strategy was developed to make the lagoon footprint available for construction. This wet weather strategy includes treatment of any flows exceeding 24 mgd, which is 2Q of the AA flow during construction. Based on historical flows at the MCR WWTP from 2015 to 2019 — as shown in Table 3-4 — it is anticipated that, on average, there will be 14 days per year that exceed 2Q.

Table 3-4 Summary of Historical Daily Average Influent Flows

YEAR	<1.25Q	1.25-1.75Q	1.75-2.00Q	2.00-2.25Q	2.25-2.75Q	2.75-4.5Q	4.5-6Q
2015	298	39	14	11	1	2	0
2016	310	31	13	7	2	1	2
2017	317	31	4	3	4	3	3
2018[1]	322	25	6	3	3	4	2
2019[2]	309	29	7	5	5	9	1
Average	311	31	9	6	3	3	2

- No flow data from 3/30-5/7/2018. Gap filled in with 2017 data.
- Data provided through Aug. 2019. Remainder of year filled in with 2018 data.

During construction, when flows exceed 24 mgd, flow will be sent directly to lagoon Cell 8. To get wet weather flows to Cell 8, some piping modifications are required. Currently at MCR, flows pumped from the wet weather pumps in the IPS send flow to the Drop Box Structure located southwest of Cell 3. The drop box structure sends flow to Cells 3 and 4 via a 72-inch reinforced concrete pipe header. Effluent from Cells 5 and 6 are also collected in a 72-inch RCP header before flowing into Cell 8. The plan to connect the IPS directly to Cell 8 involves installing approximately 200 feet of 72-inch RCP piping to connect the existing header for Cells 3 and 4 with the existing header for Cells 5 and 6. This piping connection is approximately 10 feet below the existing grade. The depth of this connection increases the complexity of construction since this will likely include an open cut excavation; however, this approach is the most cost effective due to the shortest length of large diameter piping. Other alternatives included installing much longer runs of large diameter piping and require some temporary pumping. A flow schematic for getting flows to Cell 8 is shown in Figure 3-1 Wet Weather Flows to Cell 8 Options. The selected route is Option B, which is shown in red.

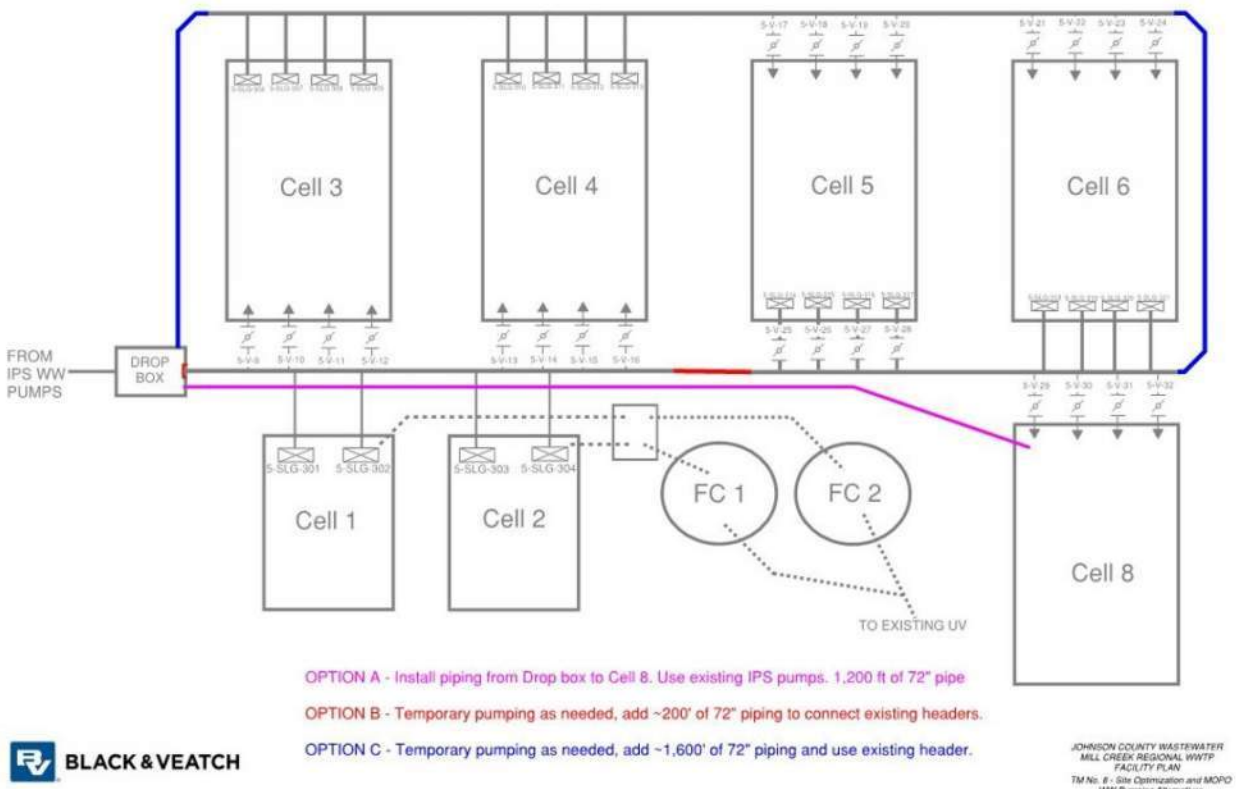


Figure 3-1 Wet Weather Flows to Cell 8 Options

Wet weather flows need to obtain some level of treatment prior to discharging into the effluent tunnel and Kansas River. Since Cell 8 is the only existing connection to the effluent tunnel, it makes sense to send wet weather flows directly to Cell 8. Lagoon effluent from Cell 8 gets to the tunnel as it flows over one of three morning glory weirs, into the Plant Effluent Junction Box. The elevation of the weirs is fixed at an elevation of 781.00 ft. This is seven feet above the bottom of Lagoon Cell 8. Other lagoon cells have a SWD of approximately 12 feet, so the volume of Lagoon Cell 8 is just under 50 percent when compared to the other lagoon cells. This reduced SWD was used when calculating storage volumes and when modeling wet weather effluent of Cell 8; however, it should also be noted that there is the potential for more volume in Cell 8 if desired by JCW. If JCW wants to increase the volume of Lagoon Cell 8, stop logs can be installed in the Plant Effluent Junction Box. Installing four stop logs in the Plant Effluent Junction Box increases the level in Cell 8 to 781.92 ft.

Cell 8 is expected to provide some solids removal and contact time for chlorination of wet weather flows. The degree of solids removal is expected to be variable, depending on the sludge blanket depth and degree of mixing. Adding baffles to Cell 8 is recommended to maximize solids removal by minimizing mixing where flow comes into Cell 8. At peak conditions, the entrance velocity could be up to 6.6 FPS. A "jersey barrier," or a series of energy dissipating blocks in the bottom of the lagoon, would be an effective solution to protect the basin sludge blanket from high velocity influent. The dimensions of Cell 8 result in a peak surface overflow rate of approximately 450 gallons per day per square foot (gpd/sf). When comparing this value to a primary clarifier, this value is on the lower end (confirming this approach as a practical solution).

Existing aspirating mixers will be moved to Cell 8 and will be run continuously during dry weather conditions to provide an oxic cap. These aspirating mixers will minimize the odors associated with stagnant water in Cell 8. During a wet weather event, the mixers will be turned off and the influent will be dosed with sodium hypochlorite at the Drop Box Structure. Cell 8 volume provides sufficient retention time (i.e., 2.57 hours of retention time under a peak flow of 84 mgd). Sodium Bisulfite for dechlorination will be added at the Plant Effluent Junction Box.

To predict the Cell 8 lagoon effluent quality and the blended effluent quality, a mass balance was performed on the facility. The mass balance approach is explained in the following text and summarized in Figure 3-2.

- Historical flow data at MCR were provided for 2015-2019 as average daily flows (ADF). The ADF peaking factor was calculated for each day over the 2015 – 2019 period by dividing the ADF by the AA flow rate. This daily ADF peaking factor was then multiplied by the projected AA flow during MCR WWTP Expansion construction (i.e., 12 mgd) to predict the future ADF during the construction period. Note that any missing ADF data was filled with corresponding values from the previous year (e.g., flow data was provided through Aug. 2019; the remainder of the year was completed with Sept.-Dec. 2018 data).
- Once the predicted flows were calculated, they were used to determine the daily influent total suspended solids (TSS), biological oxygen demand (BOD), and ammonia (NH_4-N) concentrations and loads sent to the mechanical plant and Cell 8.
  - Influent flows ≤ 24 mgd are treated by the mechanical plant. In the mass balance, influent TSS, BOD, and NH_4-N concentrations and loads to the mechanical plant were irrelevant as the mechanical plant effluent concentrations were set. The mechanical plant effluent concentrations were set equal to the 90th percentile of the mechanical plant 2015 - 2019 effluent data, as provided below:
    - BOD = 12.2 mg/L
    - TSS = 12.4 mg/L
    - NH_4-N = 1.1 mg/L
  - Influent flows > 24 mgd are directed to Cell 8. During construction, ADF influent flows exceeding 24 mgd are expected to be wet weather events. In the mass balance, each wet weather flow event assumed maximum month (MM) influent loads, provided below:

        - BOD = 26,890 lb/d
        - TSS = 36,460 lb/d
        - Total Kjeldahl Nitrogen (TKN) = 5,200 lb/d
        - NH_4-N = 2,810 lb/d
    - The loads were split between Cell 8 and the mechanical plant according to their respective flows.
    - It should also be noted that all influent TKN was assumed to be converted to ammonia in Cell 8. TKN is the sum of organic nitrogen and ammonia, and it is uncertain how much organic nitrogen will be converted to ammonia in Cell 8. Assuming all organic nitrogen is converted to ammonia is a conservative assumption. Lastly, the mass balance assumed no removal of BOD or nitrogen across Cell 8. Both 0 and 50 percent TSS removal rates were modeled.
- The blended effluent concentrations and flows were calculated using the mechanical plant and lagoon Cell 8 effluent.

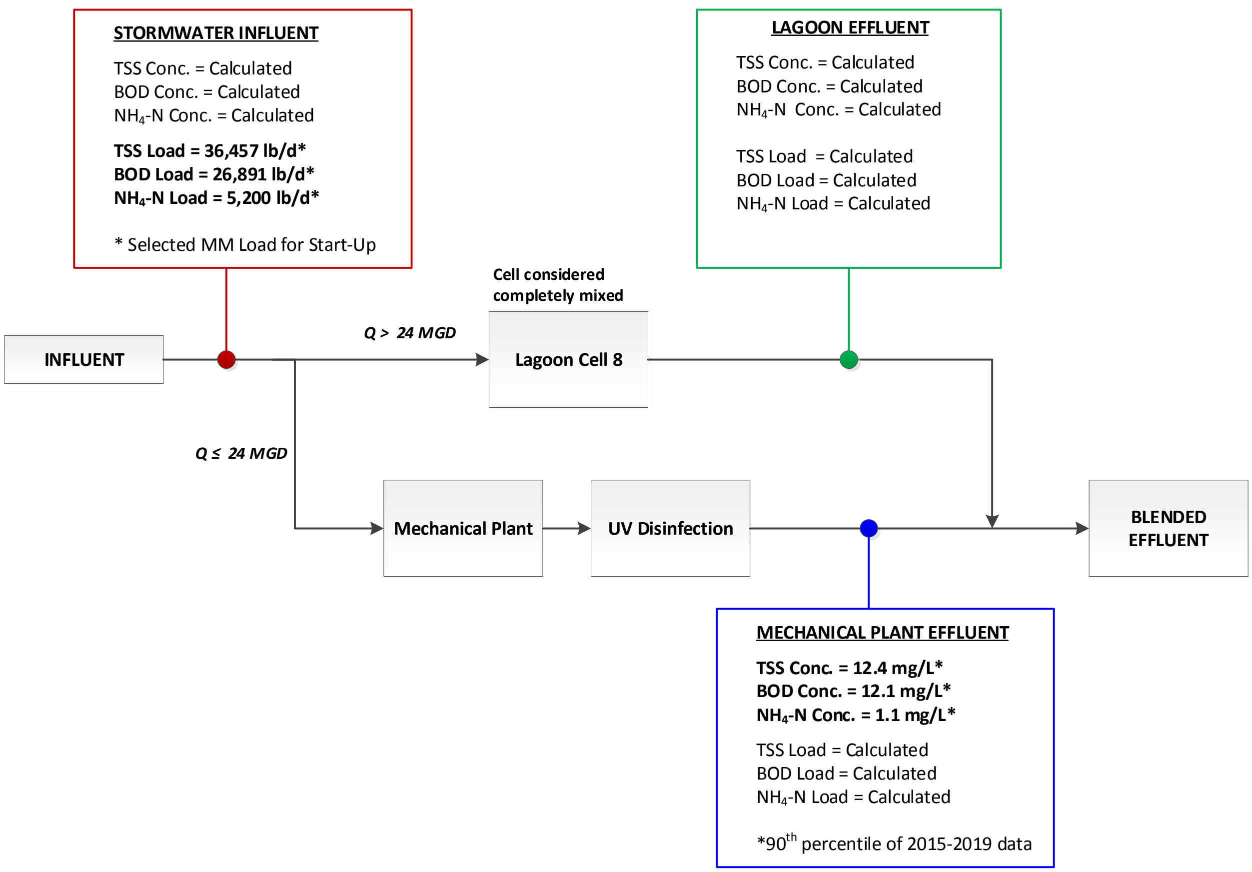


Figure 3-2 Mass Balance Model Schematic

According to the interim permit limits, TSS will be regulated separately in the mechanical plant and lagoon effluent. The mass balance modeling results are presented for lagoon effluent only in Figure 3-3.

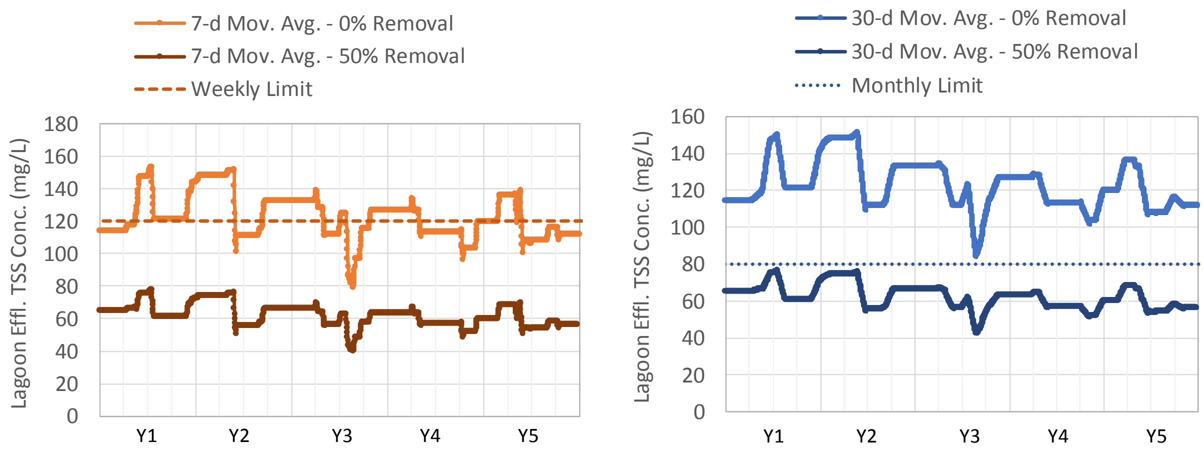


Figure 3-3 Lagoon Effluent TSS Concentrations

Assuming 0 percent removal of TSS in Cell 8, the weekly limits were exceeded approximately 50 percent of the simulated time (i.e., 2.5 out of 5 years), and the 30-day running average TSS concentration never met the monthly limit; however, with 50 percent TSS removal across Cell 8, the weekly limit was easily met. The 30-day running average modeling results also met the monthly limit, albeit with a smaller margin of safety. Note that the results shown in Figure 3-3 are the weekly and monthly average concentrations in the lagoon. Flow to the lagoon is only expected during wet weather and possibly diurnal peak hour flows. As shown in Table 3-4, historical data suggests the daily average flow will exceed 24 mgd between 12-20 times per year. Hourly exceedances will be more frequent due to wet and possibly dry weather. During AA conditions, a dry weather hourly flow exceeding 24 mgd is not expected, however there may be 1-2 hours that exceed 24 mgd during a dry weather maximum month condition. The dry weather events are expected to be very small in volume on a daily basis compared to the volume of Cell 8.

The interim NPDES permit will regulate BOD in the blended effluent. With an assumption of 0 percent removal through Cell 8, the weekly and monthly limits were easily met in the model, as shown in Figure 3-4. Similarly, ammonia will be permitted on a monthly basis in the blended effluent, with the permit limit variable from month-to-month. As shown in Figure 3-5, the simulated blended effluent ammonia concentration was maintained well under the strictest monthly limit of 14.6 mg/L (which occurs in August).

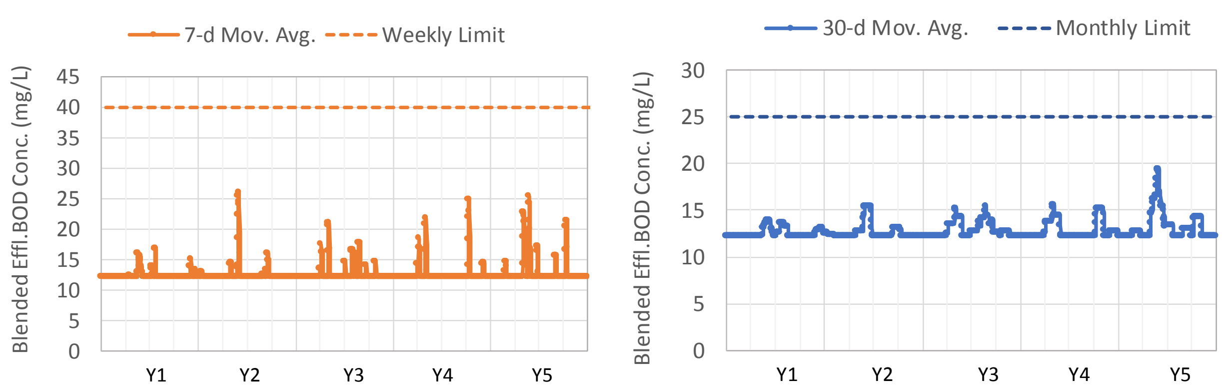


Figure 3-4 Blended Effluent BOD Concentrations

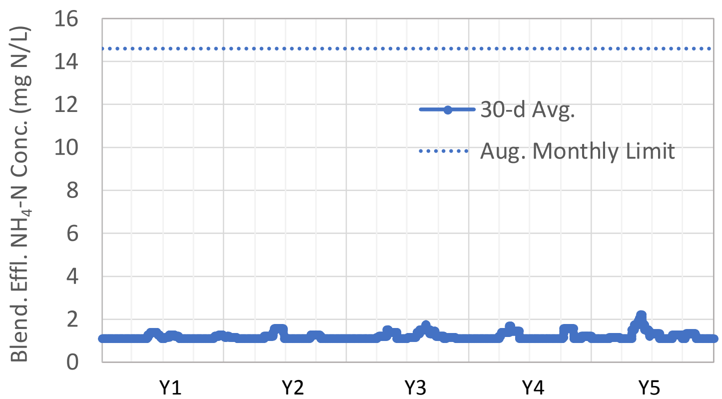


Figure 3-5 Blended Effluent Ammonia Concentration

TSS is expected to collect at the bottom of Cell 8 and will require cleaning on a regular basis. The mass balance model was used to predict the accumulation of solids in Cell 8 assuming: 1) 50 percent removal of solids, 2) 12-18 percent solids concentration in the sludge blanket, and 3) a baseline sludge blanket of 1 foot. As shown in Figure 3-6, the evaluation suggests Cell 8 will require cleaning approximately once per year but is dependent on the number of wet weather events. Looking at Figure 3-6, Y1 is based on 2015 historical data and Y5 is based on 2019 data. 2019 had many more wet weather events, so, as expected, this would result in more solids build up in Cell 8. To be able to clean Cell 8 without draining, it is recommended to use the floating barge approach that has been used in the past at the MCR WWTP. This method can effectively reduce the sludge blanket depth to approximately 1 foot, which is why Figure 3-6 shows the baseline sludge blanket as 1 foot.

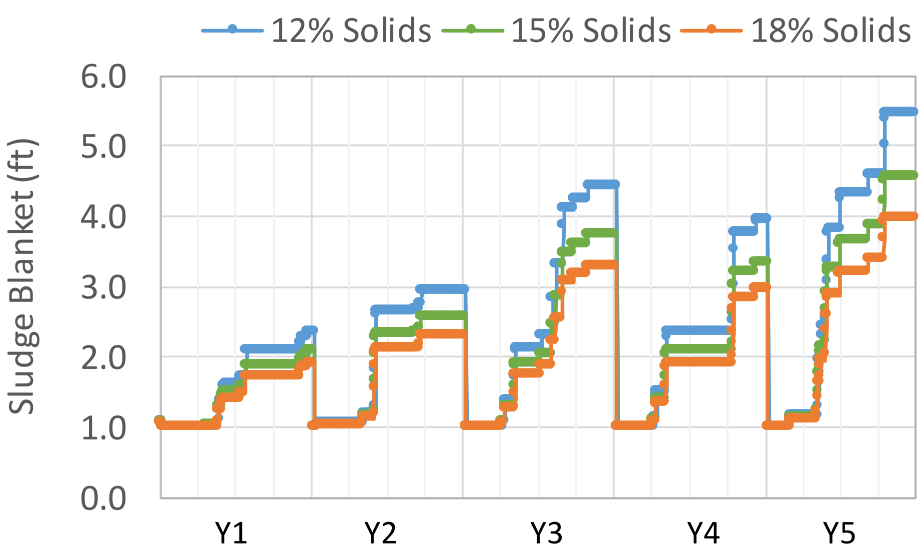


Figure 3-6 Accumulation of Solids in Cell 8 for Wet Weather Treatment

3.3 SOLIDS HANDLING STRATEGY

Since the MCR WWTP Expansion layout will be in the footprint of several existing lagoon cells, it was important to develop a strategy to handle the solids produced during construction without using several of the lagoons. Multiple options were discussed, including mobile dewatering and land application, thickening and hauling to other facilities, and dewatering with lime stabilization; however, the recommended solids handling strategy is to use Cell 6 for waste activated sludge (WAS) storage and stabilization then clean out as needed. It is believed that this is the most cost-effective alternative. JCW is familiar with the lagoon cleanout process as the current practice at the MCR WWTP is to clean out the lagoon cells on an annual basis. JCW has also developed relationships with contractors over the years who can do this work. The biggest question about this approach is to confirm the cleaning frequency.

To help determine the anticipated cleaning frequency, the 2019 lagoon cleanout historical data was reviewed. The average sludge blanket depth was 3.7 feet, and the average sludge thickness was 18 percent. Although JCW has started to clean out the lagoon cells annually, this was not always the practice. As such, it is believed that this solids concentration may be higher than what percent solids could be achieved in Cell 6, but this is still a good data point. Figure 3-7 presented below shows available storage in Cell 6 based on varying percent solids and sludge blanket depths. It should be noted that Cell 6 has a SWD of 12 feet.

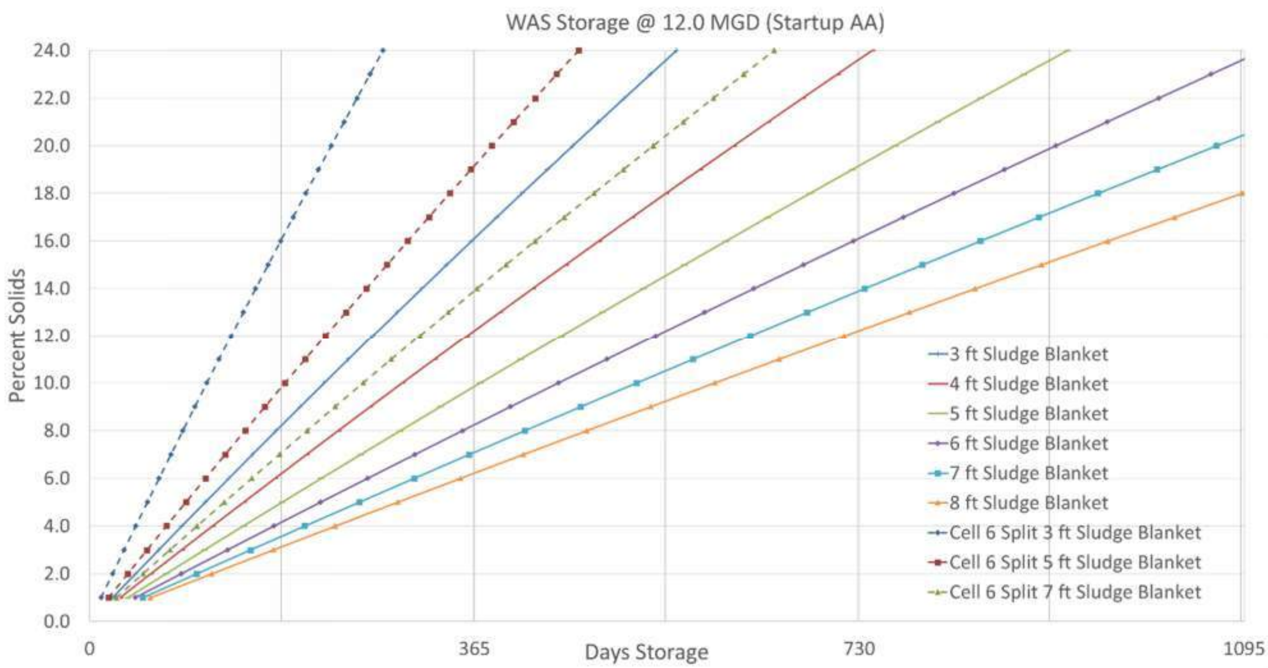


Figure 3-7 Cell 6 Sludge Storage

Looking at Figure 3-7 and using the historical data from the 2019 cleaning, an estimated storage can be determined. At 18 percent solids and a sludge blanket of approximately 4 feet, Cell 6 would have roughly a year and a half of storage. Another example is 10 percent solids with a 5-foot sludge blanket in Cell 6, which would result in approximately one year of storage. JCW is comfortable with annual lagoon cleanouts, so managing the WAS to this schedule is a suitable solution.

Historically, when JCW cleans out a lagoon, the cell is drained to the extent possible and the Contractor is allowed to work until the solids have been removed. When Cell 6 is the only operating lagoon cell for WAS storage, this approach will not work. One solution to continue storing solids while cleaning Cell 6 is to divide into two independent half-cells by building an East/West dividing berm or wall. The barrier could be constructed by adding sheet piles, or an earthen berm. One cell could be in a filling phase while the other would be in a stabilization/cleanout phase. This setup is also shown in Figure 3-7 by the dashed lines. For example, the same conditions of 10 percent solids with a 5-foot sludge blanket results in 6 months of storage. Overall, this results in twice as many cleanouts, but perhaps an easier cleaning experience. Operating two half-cells allows JCW to continue the storing process while the other half cell is being cleaned out. If JCW is concerned with the frequency of cleaning, it is recommended to increase the sludge blanket depth. Currently, the high-water depth in Cell 6 is 12 feet, so sludge blanket depths of 8 feet could be realistic to increase storage.

To get the WAS to lagoon Cell 6, some piping modifications will be needed. The plan is to tie into the existing 6-inch WAS discharge line near the northeast corner of Cell 2, then route to Cell 6 as needed. While it is desired to use Cell 6 for sludge storage and stabilization, the existing solids infrastructure associated with Cells 1 and 2 will remain in operation. The RAS will continue to be recycled back to lagoon Cells 1 and 2 from the Sludge Pump Station. The WAS will be able to be recycled to Cells 1 and 2, sent to the WAS loadout facility, or sent to Cell 6.

Historically, the lagoons have had odor associated with the seasonal turnover. To mitigate this, it is recommended to install six Aqua Jet Surface Mechanical Aerators in Cell 6. This will help mitigate odors associated with seasonal lagoon turnover. To install the aerators, concrete mooring posts around the edge of the lagoon will be required. To stabilize and prevent the aerators from moving laterally, cables will be attached to each aerator from three different posts on the shore.

During construction of the MCR WWTP Expansion, it is recommended to use Cell 6 for solids handling and decanting to the mechanical plant to keep an aerobic cap. Concurrently, Cell 6 should be used to balance the sludge blanket depth and solids concentration as needed to maintain annual cleaning.

3.4 MOPO COSTS

During the construction of the MCR WWTP Expansion, there are several costs that need to be captured that are associated with the ability to provide treatment during construction. These costs are not captured elsewhere and are associated with equipment that needs to be installed for the interim construction period only. Equipment listed here will not be used once construction of the MCR WWTP Expansion is completed. The following is a summary of what the costs in Table 3-5 include.

- Cell 8 Wet Weather Treatment
  - 200 feet of 72-inch RCP at approximately 15 feet below grade, and associated excavation and backfill.
  - Costs associated with disinfection of Cell 8. This includes capital costs associated with dosing 10 mg/L of sodium hypochlorite in the drop box structure during dry weather treatment, and dosing sodium bisulfite at the Plant Effluent Junction Box for dechlorination. Includes storage tanks, feed pumps, and piping.

- O&M Costs associated with chemical dosing based on historical wet weather events greater than 2Q as shown in Table 3-4.
- Relocation cost for moving several existing MCR WWTP aspirating mixers to Cell 8, and a cost for a "jersey barrier," or a baffle system at the entrance of Cell 8.
- Annual cleaning of Cell 8 based on average conditions from Figure 3-6

■ Cell 6 Solids Handling

- 1,200 feet of 6-inch DIP for sending WAS to Cell 6, and associated excavation and backfill.
- Building an earthen berm in Cell 6 to separate into two different lagoon cells.
- Six (6) five-hp Aqua-Jet Aerators and associated components installed in Cell 6 to provide an oxic cap and attempt to mitigate odors associated with seasonal turnover.
- Annual cleaning of each individual cell, assuming a 7-foot sludge blanket at 14 percent solids.

Table 3-5 MOPO Costs

SERVICE	LUMP SUM COST ($)
Cell 8 Wet Weather Treatment Costs	
Piping, excavation, backfill	400,000
Chemical capital costs	260,000
Chemical O&M costs	120,000
Mixer relocation cost	40,000
Annual cleaning	1,000,000
Cell 6 Solids Handling Costs	
Piping, excavation, backfill	200,000
Earthen partition berm	50,000
Aqua-Jet mixers	60,000
Mixer power and mixer maintenance cost	40,000
Annual cleaning	2,400,000
Total	4,570,000

- Cell 8 cleaning assumes two total cleanings, assumes filter complex comes online prior to end of construction
- Cell 6 cleaning assumes four total cleanings
- Costs are presented in January 2020 dollars

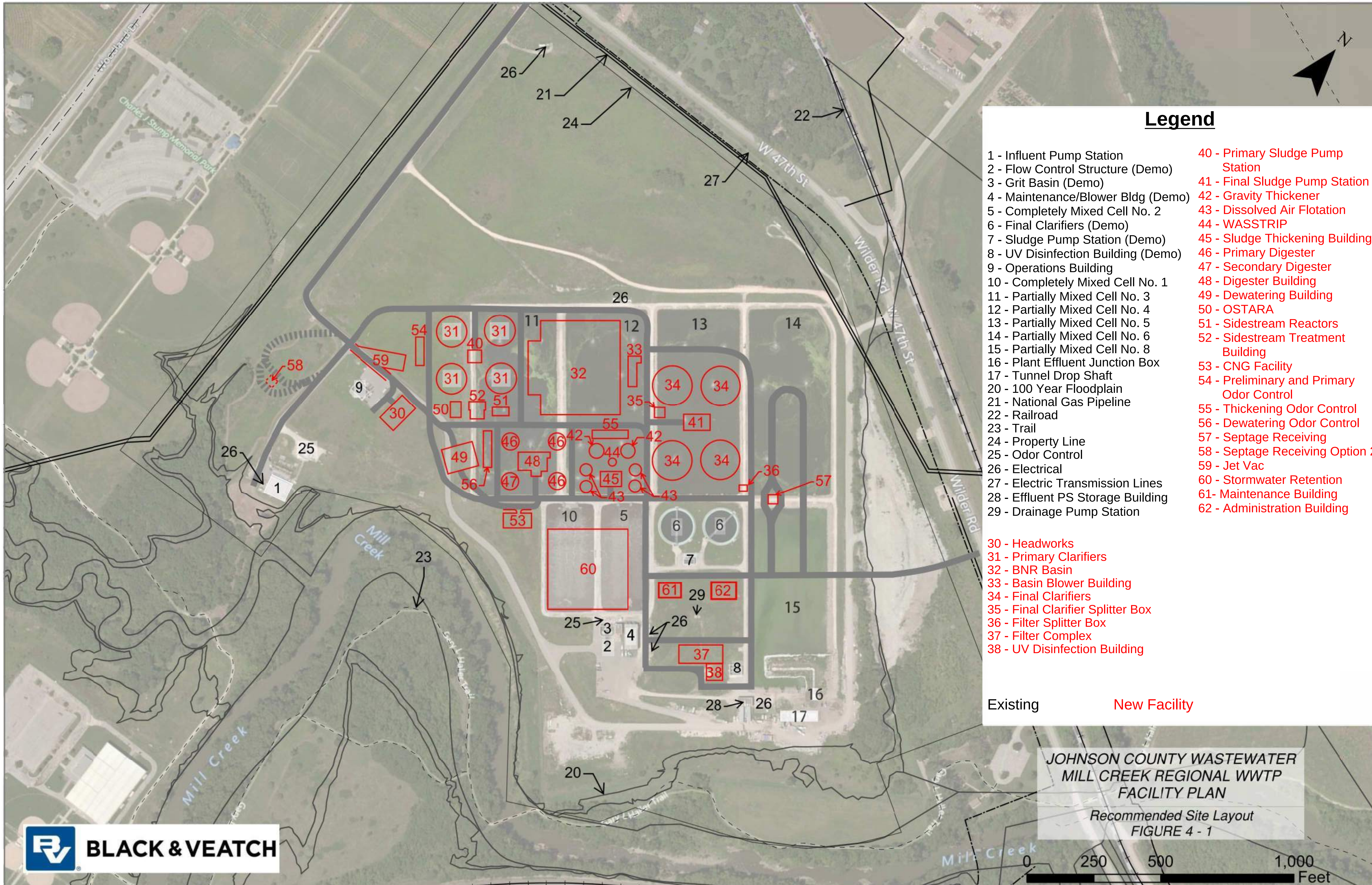
4.0 Summary of Findings and Recommendations

MCR WWTP has a 100-year flood elevation of approximately 773.0 ft., and a 500-year flood elevation of approximately 779.0 ft. The significant increase in elevation associated with the 500-year elevation is due to a backwater effect from the Kansas River. Most of the existing site, including all existing facilities are above the 500-year flood elevation. Given the availability of land, all new facilities will also be located above the 500-year floodplain elevation.

Preliminary site layouts determined that the location of the solids processing facilities and the location of the Filter Complex and UV Disinfection facility were important considerations. It was determined that the location of the solids processing facilities should be central to the MCR WWTP Expansion. A central location will make these facilities less visible and less noticeable to the surrounding recreational sports complex and walking trail. Three different locations were evaluated for the Filter Complex and UV Disinfection facility. In the end, because of the constructability and the estimated cost benefit it was recommended to locate these facilities in the former lagoon Cell 7. The recommended site layout for the MCR WWTP Expansion is shown in Figure 4-1.

While it is anticipated that interim and ultimate IPS improvements will be required to meet the predicted flows to MCR, based on preliminary hydraulic modeling it was determined that a pump once profile can be achieved for the MCR WWTP Expansion. As such, the MCR WWTP Expansion does not include intermediate pumping. Once flows are pumped to the Headworks Building, they will flow by gravity to the effluent discharge tunnel.

It is important to understand that during construction of the MCR WWTP Expansion, treatment of the daily average and wet weather flows to meet permit limits is required. It is equally important to develop a plan to handle the solids that are produced during this construction period. The recommended MOPO includes using the existing mechanical plant for flows less than 24 mgd, lagoon Cell 8 for wet weather treatment, lagoon Cell 6 for WAS solids storage and stabilization.


Legend
1 - Influent Pump Station
2 - Flow Control Structure (Demo)
3 - Grit Basin (Demo)
4 - Maintenance/Blower Bldg (Demo)
5 - Completely Mixed Cell No. 2
6 - Final Clarifiers (Demo)
7 - Sludge Pump Station (Demo)
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9 - Operations Building
10 - Completely Mixed Cell No. 1
11 - Partially Mixed Cell No. 3
12 - Partially Mixed Cell No. 4
13 - Partially Mixed Cell No. 5
14 - Partially Mixed Cell No. 6
15 - Partially Mixed Cell No. 8
16 - Plant Effluent Junction Box
17 - Tunnel Drop Shaft
20 - 100 Year Floodplain
21 - National Gas Pipeline
22 - Railroad
23 - Trail
24 - Property Line
25 - Odor Control
26 - Electrical
27 - Electric Transmission Lines
28 - Effluent PS Storage Building
29 - Drainage Pump Station
30 - Headworks
31 - Primary Clarifiers
32 - BNR Basin
33 - Basin Blower Building
34 - Final Clarifiers
35 - Final Clarifier Splitter Box
36 - Filter Splitter Box
37 - Filter Complex
38 - UV Disinfection Building
40 - Primary Sludge Pump Station
41 - Final Sludge Pump Station
42 - Gravity Thickener
43 - Dissolved Air Flotation
44 - WASSTRIP
45 - Sludge Thickening Building
46 - Primary Digester
47 - Secondary Digester
48 - Digester Building
49 - Dewatering Building
50 - OSTARA
51 - Sidestream Reactors
52 - Sidestream Treatment Building
53 - CNG Facility
54 - Preliminary and Primary Odor Control
55 - Thickening Odor Control
56 - Dewatering Odor Control
57 - Septage Receiving
58 - Septage Receiving Option 2
59 - Jet Vac
60 - Stormwater Retention
61- Maintenance Building
62 - Administration Building
Existing
New Facility
JOHNSON COUNTY WASTEWATER
MILL CREEK REGIONAL WWTP
FACILITY PLAN
Recommended Site Layout
FIGURE 4 - 1
0
250
500
1,000
Feet
N
W 47th St
Wilder Rd
Mill Creek
BLACK & VEATCH

DRAFT

MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 9
Influent Pumping

JCW NO. MCR1-BV-17-12
B&V PROJECT 403165

PREPARED FOR

JOHNSON COUNTY KANSAS
Wastewater

OCTOBER 12, 2020

BLACK & VEATCH

Table of Contents

LIST OF TABLES

LIST OF FIGURES

Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CANDO	Coupled Aerobic-anoxic Nitrous Decomposition Operation
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEA	Cost Effective Analyses
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
CIPP	Cured-in-place Pipe
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSBR	Continuous Sequencing Batch Reactor
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
D	
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry-weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand

Abbreviation	Meaning
ffCOD	Flocculated Filtered Chemical Oxygen Demand
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
ft	Feet
Fps	Feet per Second
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
gpcd	Gallons per capita per day
gpd	Gallons per Day
gpm	Gallons per minute
H	
HB	Hallbrook Facility
HDD	Horizontal Directional Drilling
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration
IC	Internal Combustion
IFAS	Integrated Fixed-Film Activated Sludge
in	Inches
IND	Industrial

Abbreviation	Meaning
INF	Influent
IP	Intellectual Property
IPS	Influent Pump Station
IR	Irrigation Use
IRR	Irrigation
IW	Industrial Water Supply Use
J	
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDHE	Kansas Department of Health and Environment
K_e	Light Extinction Coefficient
kWh	Kilowatt-Hour
L	
L	Length, Liter
lb	Pound
LF	Linear Feet
LOMR	Letter of Map Revision
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter
mgd	Million Gallons per Day
min	Minute, minimum
mJ	Millijoules
MLE	Modified Ludzack Ettinger

Abbreviation	Meaning
MLSS	Mixed Liquor Suspended Solids
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow
mmBtu	Million British Thermal Units
MOPO	Maintenance of Plant Operations
mpg	Miles per Gallon
MPN	Most Probable Number
µg/L	Micrograms per Liter
N	
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NFIP	National Flood Insurance Program
NH3-N	Total Ammonia
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
NPV	Net Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow
PE	Primary Effluent
PFE	Primary Filtered Effluent
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow
PIF	Peak Instantaneous Flow
PLC	Programmable Logic Controller
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day
pph	Pounds per Hour
PPI	Producer Price Index
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet-Weather Flow
Q	
Q	Flow
R	
RAS	Return Activated Sludge
RAS	
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
Sec	Second, Secondary
SF	Square Foot
SG	Specific Gravity
SLR	Solids Loading Rate
SMP	Stormwater Management Program, Shawnee Mission Park Pump Station
SND	Simultaneous Nitrification/ Denitrification

Abbreviation	Meaning
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station
SRT	Sludge Retention Time
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
TYP	Typical
U	
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VFA	Volatile Fatty Acids
VFAs	
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	Width
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation –Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WL	Water Level
WK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

The purpose of this technical memorandum (TM) is to summarize the conceptual design of both the influent and offsite pumping at the Mill Creek Regional (MCR) wastewater treatment plant (WWTP). This TM includes a discussion of the existing pumping infrastructure, recommended improvements to account for interim and ultimate influent pumping at the MCR WWTP, and an investigation of the offsite pump stations at ultimate flow conditions.

This TM is one in a series of technical memoranda for the MCR Facility Plan. Additional treatment processes, optimization, and implementation of these facilities are outlined in other TMs.

Wastewater from the Mill Creek watershed flows primarily by gravity through a sanitary sewer network which ends with the Mill Creek Interceptor flowing into the Influent Pump Station (IPS) at the MCR WWTP. At the IPS, influent is screened and pumped to the Flow Control Structure. At the Flow Control Structure, the IPS effluent is combined with flows from three different offsite pump stations: Cedar Mill, Tooley Creek, and 55th Street. A map showing the locations of each of these pumping stations and the MCR WWTP is presented in Figure 1-1.

Figure 1-1 MCR WWTP and Offsite Pump Stations

One of the objectives of this TM is to determine if any improvements are recommended at each of the three offsite pump stations. As part of the MCR WWTP Expansion, the Flow Control Structure will be demolished, and as discussed in TM 2 – Preliminary and Primary Treatment, a new Headworks Building will be constructed. Effluent from the IPS and flows from the offsite pump stations will be rerouted to the new Headworks building. Each of the offsite pump stations are approximately 20 years old. There are not any major issues at any of these pump stations, however, since the force main discharge locations are being revised as part of the MCR WWTP Expansion (as discussed in TM – 8 Site Optimization and MOPO), it is important to understand the associated effects at each offsite pump station.

Another objective of this TM is to determine what improvements are required at the IPS to accommodate a range of current and future flows. Given the location and depth of the IPS, it is preferable to maintain use of the existing facility as part of the MCR WWTP Expansion. To be able to continue using the existing facility for future flows, some improvements will be required. This TM summarizes those improvements for both the interim and ultimate flow conditions.

2.0 Collection System Pumping Stations

Three offsite collection system pumping stations currently convey wastewater to the MCR WWTP, independent from the IPS. Flows from Tooley Creek Pump Station (TCPS) and 55th Street Pump Station are transported via the Tooley Creek Force main, while flows from Cedar Mill Pump Station are pumped in the separate Cedar Mill Force main. Both are 16-inch ductile iron pipe (DIP) and discharge at the Flow Control Structure. After the MCR WWTP Expansion, both Force mains will be re-routed to the future Headworks Building, as shown in Figure 2-1.

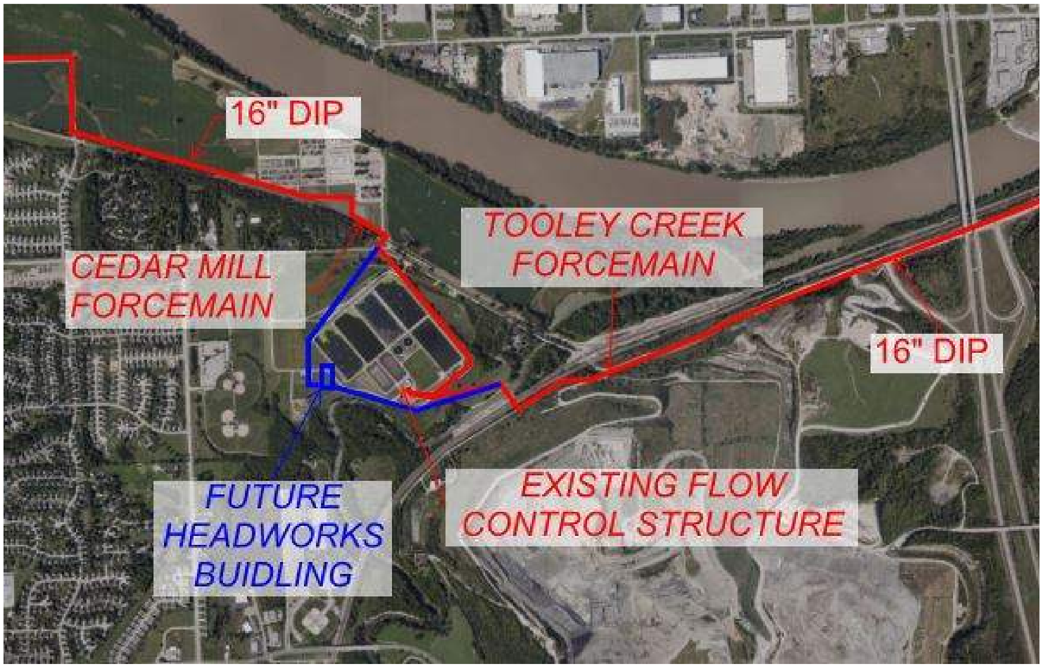

Figure 2-1 Offsite Force mains - Existing and Future Routes

Daily historical flow data was provided by JCW for all three offsite pump stations between January 2013 and September 2019. Minimum, maximum, and average daily flow rates are listed below in Table 2-1.

Table 2-1 MCR Offsite Pump Station Flow Data

	DAILY FLOW (MGD)		
	Minimum	Average	Maximum
55th Street Pump Station	0.20	0.48	3.78
Tooley Creek Pump Station	0.01	0.08	0.78
Cedar Mill Pump Station	0.30	0.66	2.92

- Based on historical daily flow data between January 2013 and September 2019.

2.1 55TH STREET PUMP STATION

55th Street Pump Station (PS) was built in 2001 next to an existing pump station at the corner of Alden Street and 55th Street. 55th Street PS consists of an electrical building and pump building. There is also a chemical storage tank on-site that is used to inject Bioxide™ into the wastestream for odor control, but it is not typically in service. Four pumps are housed in the pump building – two small and two large. The two small pumps are typically in service to pump dry weather flow. When the influent flow is greater than the combined capacity of the small pumps, a large pump starts. The design criteria for the pumps are presented in Table 2-2.

Table 2-2 55th Street PS Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Small Pumps	
Number of Units	2 (duty)
Pump Type	Wet-Pit Submersible
Rated Capacity, gpm (each)	1,100
Head at Rated Capacity, ft	170
Motor, hp (each)	100
Drive Type	Constant Speed
Large Pumps	
Number of Units	2 (1 duty, 1 standby)
Pump Type	Wet-Pit Submersible
Rated Capacity, gpm (each)	3,000
Head at Rated Capacity, ft	170
Motor, hp (each)	250
Drive Type	Constant Speed
Pump Station Firm Capacity, gpm	5,600

From 55th Street PS, wastewater is pumped via an 18-inch DIP main to the Tooley Creek Force main. The 18-inch main is routed approximately 12,400 linear feet (lf) before it reduces to 16 inches and connects to the Tooley Creek Force main. A schematic showing the two pump stations is shown in Figure 2-2.

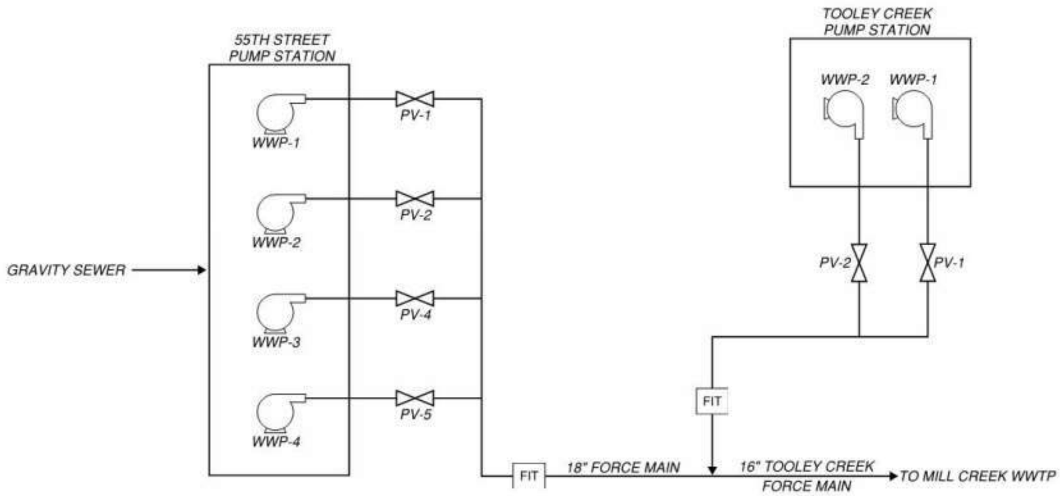

Figure 2-2 Schematic of 55th Street PS and TCPS

The Tooley Creek Force main is approximately 12,000 lf and currently discharges at the Flow Control Structure at the MCR WWTP. After the MCR WWTP Expansion, the Tooley Creek Force main will be extended to discharge at the new Headworks Building. According to the site layout recommended in TM 8 – Site Optimization and MOPO, the Headworks Building will be located near the existing Operations Building. A comparison of the existing and future force main route is presented in Table 2-3.

Table 2-3 Tooley Creek Forcemain Comparison

	EXISTING	FUTURE
Length, ft	12,000	13,000
Discharge Elevation, ft	793.5	807.3

Because TCPS and 55th Street PS tie into the same force main, the pumps at each are impacted by the flow at the other pump station. To evaluate the pumps at 55th Street PS, a constant flow for TCPS was assumed. Historical flow data indicates that the historical maximum daily flow at TCPS is 0.78 million gallons per day (mgd), or 650 gallons per minute (gpm). System curves were modeled for existing and future conditions using the maximum historical daily flow at TCPS. Pump and system curves for 55th Street PS are shown in Figure 2-3. In addition, Figure 2-3 shows future maximum and minimum conditions at 55th Street PS if the TCPS pumps are not in operation.

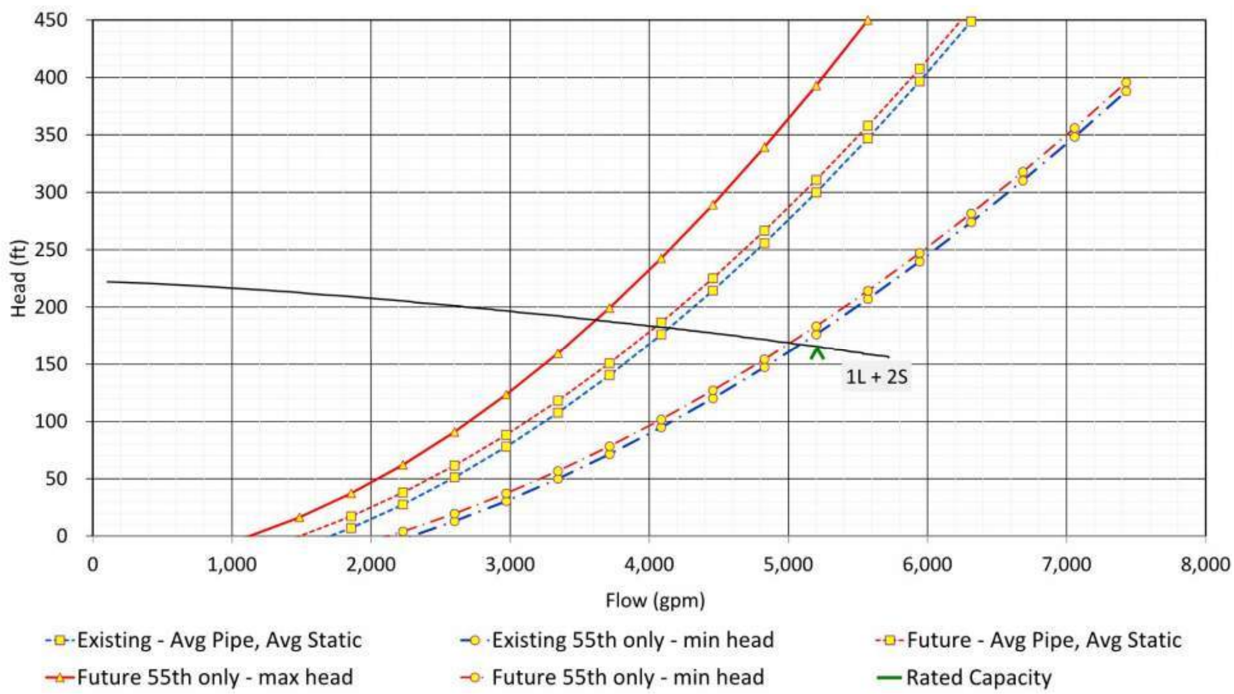

Figure 2-3 55th Street PS Pump Curve (TCPS @ Max Historical Daily Flow)

After the MCR WWTP Expansion, during normal conditions (TCPS is in operation) there will be slightly more headloss for the pumps at 55th Street PS to overcome. Adding headloss to the system will move the system curve closer to the rated and best efficiency point for all conditions. The duty pumps (two small pumps and one large pump) will still be able to adequately pump the max historical daily flow of 3,150 gpm. If TCPS is offline, and 55th Street is at minimum conditions, the system headloss will be decreased when compared to the average conditions; however, when the future minimum conditions are compared to the existing minimum conditions, there is still a net increase in headloss. Because the future conditions are an improvement over the existing conditions at all flows, the only upgrade recommended for 55th Street PS is pump replacement once the equipment reaches the end of its useful life.

2.2 TOOLEY CREEK PUMP STATION

Tooley Creek Pump Station is located on S. 85th Street near Holliday Drive. The pump station was originally constructed with a wetwell, headworks building, chlorine contact basin, and two storm water holding basins. All treatment facilities were decommissioned in 1993 and a force main was constructed to convey flow from TCPS to the MCR WWTP for treatment. In 2001, 1 of the 3 pumps in the wetwell was removed and two (2) 980 gpm submersible pumps replaced the previous pumps. Design criteria for these pumps is presented in Table 2-4.

Table 2-4 Existing TCPS Pump Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	2 (1 duty, 1 standby)
Pump Type	Wet-pit Submersible
Rated Capacity, gpm (each)	980
Head at Rated Capacity, ft	147
Motor, hp (each)	100
Drive Type	AFD

As described in Section 2.1, 55th Street PS discharges to the Tooley Creek Force main and the combined flow has an impact on the pumps at TCPS. To evaluate the pumps at TCPS, a constant flow rate for 55th Street PS was assumed. Historical flow data indicates that the maximum daily flow at 55th Street PS is 3.77 mgd, or 3,100 gpm. System curves were modeled for existing and future conditions using the maximum historical daily flow at 55th Street PS. Pump and system curves for TCPS are shown in Figure 2-4. In addition, Figure 2-4 shows future maximum and minimum conditions at TCPS if the 55th Street PS pumps are not in operation.

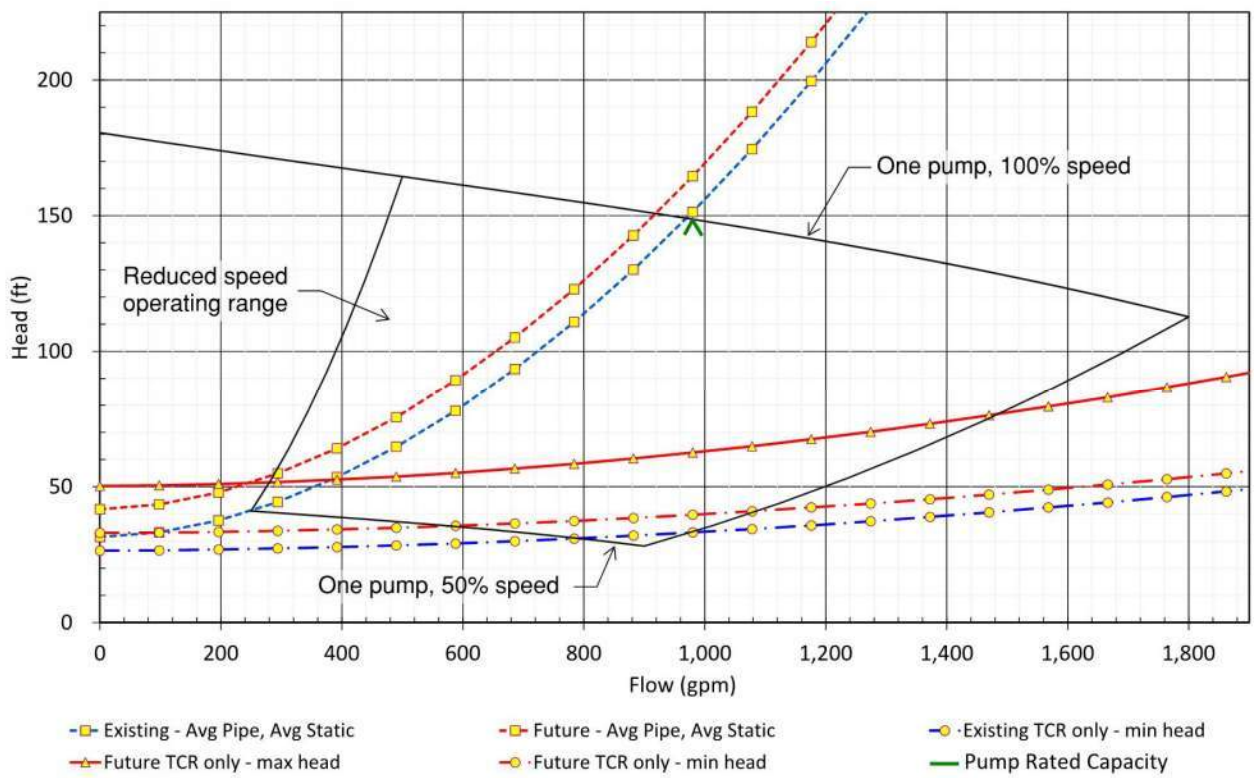

Figure 2-4 TCPS Pump Curve (55th Street PS @ Max Historical Daily Flow)

After the MCR WWTP Expansion, there will be slightly more headloss for the pumps at TCPS to overcome. Adding headloss to the system will move the system curve closer to the rated and best efficiency point at all conditions. The duty pump will still be able to adequately pump the max historical daily flow of 650 gpm. Since the future hydraulic conditions are expected to bring the system curve closer to the pump rated condition, the only upgrade recommended for TCPS is pump replacement once the equipment reaches the end of its useful life.

2.3 CEDAR MILL PUMP STATION

Cedar Mill Pump Station is located on W. 43rd Street between K-7 Highway and Lakecrest Drive. The pump station was constructed in 1995 and consists of a wetwell and a Jet-Vac truck dumping station, as described in TM 7 – Support Facilities. The wetwell is fitted with three pumps and a basket screen. Two of the pumps are original, and one pump was replaced in 2012. The design criteria of the pumps are presented in Table 2-5.

Table 2-5 Cedar Mill PS Pumps Design Criteria

PARAMETER	DESIGN CRITERIA
Number of Units	3 (2 duty, 1 standby)
Pump Type	Wet-pit submersible
Rated Capacity, gpm (each)	1,250
Rated Total Head, ft	130
Pump Station Firm Capacity, gpm	4,450
Motor, hp (each)	90
Drive Type	Constant Speed

From Cedar Mill PS, wastewater is pumped approximately 11,700 lf via a 16-inch DIP. Currently, the force main discharges at the MCR WWTP Flow Control Structure. After the expansion, the force main will be routed along the west side of the site and will discharge at the new Headworks Building. The pump and system curves for the existing and future conditions were modeled and are shown in Figure 2-5.

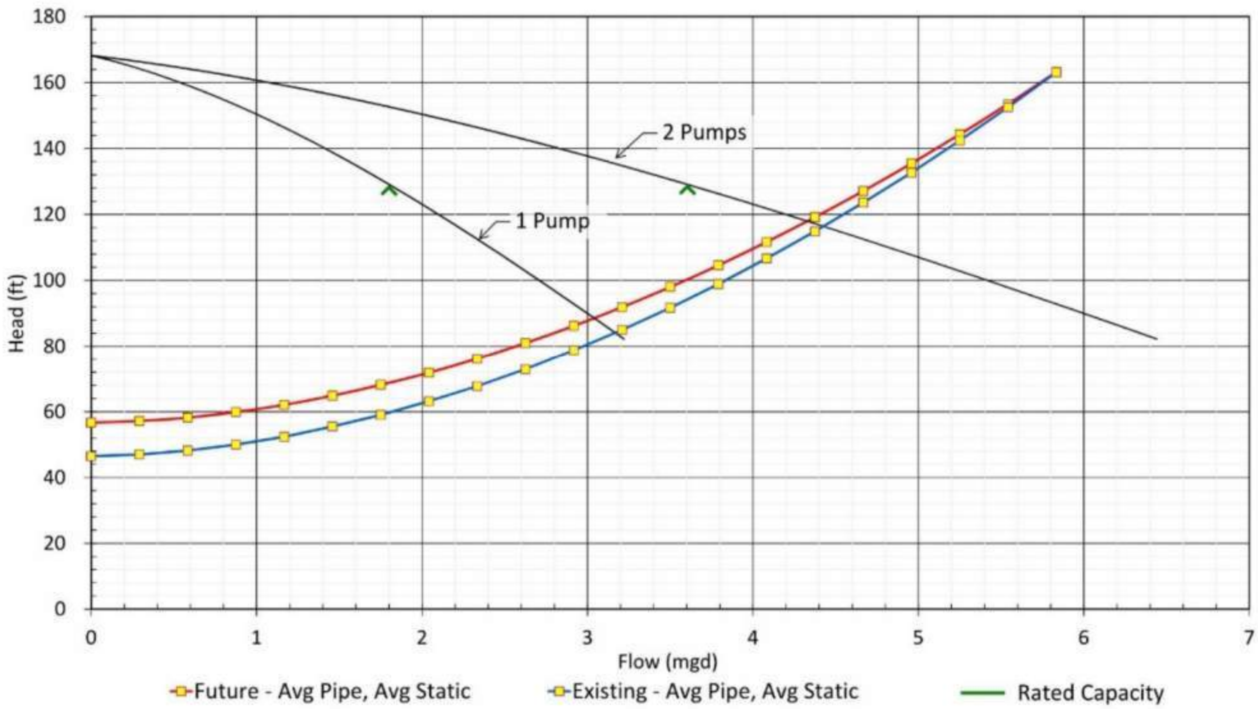

Figure 2-5 Cedar Mill PS Pump Curve

After the MCR WWTP Expansion, there will be slightly more headloss in the force main. Historical daily flow data indicates that the average daily flow at Cedar Mill PS is 0.66 mgd, or approximately 460 gpm. The maximum historical daily flow is 2.92 mgd, or approximately 2,030 gpm. Based on the design capacity, as long as flows are below around three mgd, one pump operating is sufficient. When the system is approaching maximum conditions and beyond, one pump in operation is approaching the end of the curve. Therefore, as the system approaches maximum conditions, two pumps are likely required. Overall, the slight increase in headloss will move the system curve closer to the pumps rated point.

In 2017, George Butler Associates, Inc. (GBA) conducted an evaluation of the Cedar Mill Watershed. It was estimated that approximately 70 percent of the watershed is undeveloped or serviced by septic systems. They found that the wetwell at Cedar Mill is not deep enough to service gravity lines from a portion of those remote areas. To resolve this issue, it was proposed that a new pump station be constructed on the southeast corner of K-7 Highway and 43rd Street. In addition, the equipment at Cedar Mill Pump Station is reaching the end of its useful life; thus, rehabilitation is necessary to extend the life of the pump station. For the purposes of this TM, there are no recommended improvements at Cedar Mill Pump Station; however, it is recommended that a future study confirm the findings of the GBA evaluation prior to implementation.

3.0 MCR WWTP Influent Pumping Station

Roughly 90 percent of all flow that comes to the MCR WWTP arrives at the IPS via the Mill Creek Interceptor. The Mill Creek Interceptor is a 66-inch reinforced concrete pipe (RCP) that was built in 1992. Its 57,000-foot length serves as the sewer backbone of the Mill Creek Watershed. The interceptor was originally designed and constructed to provide capacity for development through 2010. As discussed in previous TMs, it is expected that construction of the MCR WWTP Expansion will be completed by 2035. Previous TMs have focused on predicting flows at startup conditions and sizing equipment accordingly. For the IPS improvements, it is important to not only understand the ultimate flow conditions, but it is also important to be able to account for existing flows through the completion of the MCR WWTP Expansion. Throughout this TM, the period between present day and the completion of the MCW WWTP Expansion is referred to as the Interim condition.

The recommended Interim improvements at the MCR WWTP are focused on improving the wet weather pumping capacity and the screenings process, while the ultimate improvements are focused on improving the dry weather pumping capacity.

3.1 INTERIM IPS IMPROVEMENTS

Once flow enters the IPS, it flows through one of the four bar screen channels where it is screened using a mechanically-cleaned “climber” type bar screen. After the influent has been screened, flow goes into IPS Wetwell Nos. 1 or 2. The IPS has two dry weather wetwells (Wetwells No. 1 and No. 2) and one wet weather wetwell (Wetwell No. 3). The wetwells share common walls, and flows exceeding the wetwell capacity are diverted between the wetwells via an opening in the wall. A plan view of the IPS is presented in Figure 3-1. Note, that the system heads called out in Figure 3-1 are based on the modeled existing operating conditions.

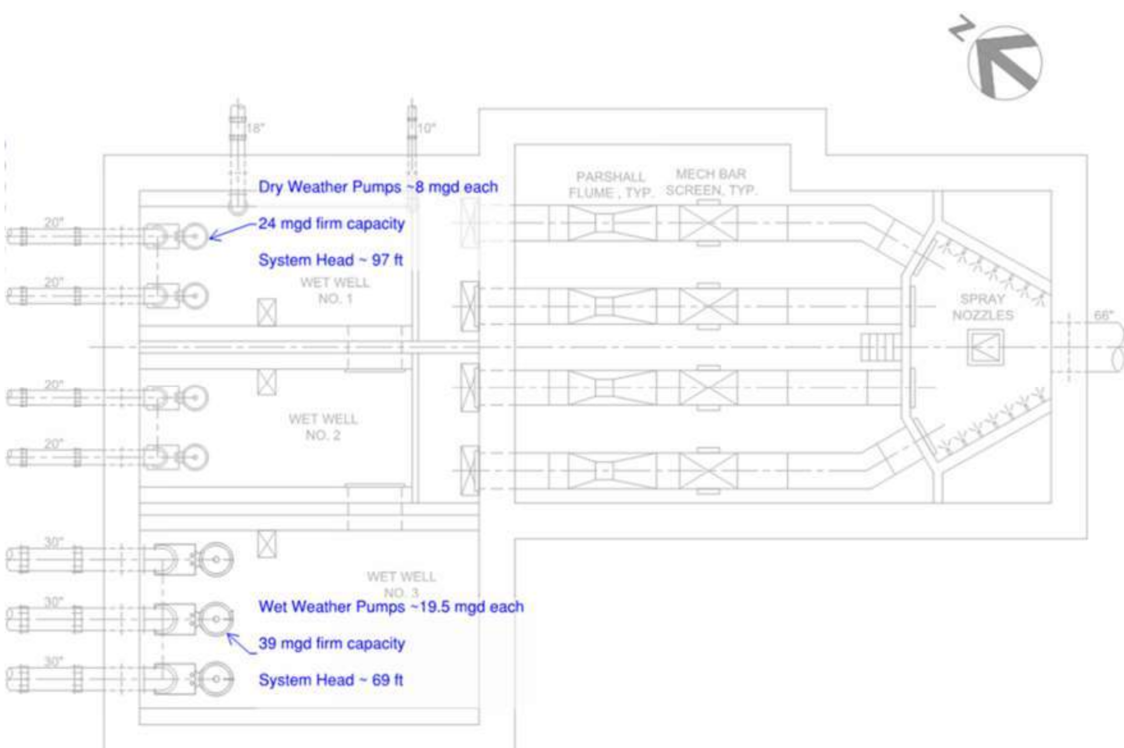

Figure 3-1 MCR WWTP IPS Plan View

There are two submersible pumps in each dry weather wetwell for a total of four dry weather pumps. These dry weather pumps were replaced as part of the most recent plant expansion in 2006. Each pump has a rated capacity of approximately eight mgd. There are three submersible pumps in the wet weather wetwell. These pumps and the bar screens are original to the plant. Each wet weather pump has a rated capacity of approximately 19.5 mgd. A summary of the existing pump station is presented in Table 3-1.

Table 3-1 Existing Influent Pumping Station Summary

COMPONENT	COMPONENT SUMMARY
Dry Weather Pumps	Number of Pumps: 4 Pump Type: Submersible, non-clog Rated Capacity, gpm (each): 5,400 Head at Rated Capacity, ft: 84 Pump Station Firm Capacity, mgd: 23.3 Motor, hp (each):190 Drive Type: Constant Speed Discharge Piping Diameter, in: 20 Wetwell Volume, cf (each): 38,900
Wet Weather Pumps	Number of Pumps: 3 Pump Type: Submersible, non-clog Rated Capacity, gpm (each): 13,500 Head at Rated Capacity, ft: 70 Pump Station Firm Capacity, mgd: 39 Motor, hp (each): 335 Drive Type: Constant Speed Discharge Piping Diameter, in: 30 Wetwell Volume, cf: 53,600

Although these wetwells are common wall construction and the discharge piping includes a common header, it is recommended to think of the dry weather wetwells and the wet weather wetwell as two separate pump stations because of the difference in system heads. There is an approximate 30-foot difference in system head between the two systems. Part of this difference is because the Dry Weather Pumping Station and Wet Weather Pumping Station discharge to different locations. Currently, the Dry Weather Pumping Station sends flow to the Flow Control Structure, where up to 24 mgd then passes through the mechanical plant. The Wet Weather Pumping Station currently discharges to the drop box structure, which is the beginning of the lagoon train. If the shared discharge header valves were oriented such that flow could go to either the Flow Control Structure or the Drop Box Structure, flow would go towards the lower system head, which is the Drop Box Structure. The existing plug valve that separates the dry weather and wet weather discharge is a manual valve. This manual valve is normally closed to allow the Wet Weather and Dry Weather Pumping Stations to operate independently. While it would be possible to replace this valve with an electric valve to provide throttled control of the pump station discharge, a throttled valve operation approach is not desired by JCW or recommended by BV at this planning level.

Although the ultimate flow improvements will be discussed later in this TM, it should be noted that, based on the preliminary hydraulics in TM 8 – Site Optimization and MOPO, it is not anticipated that the Wet Weather and Dry Weather Pumping Stations will operate with a similar system head after the MCR WWTP Expansion. The Dry Weather Pumping Station will send flow to the Headworks Building, and the Wet Weather Pumping Station will send flow to the Filter Complex for auxiliary wet weather treatment.

3.1.1 Interim Flow Summary

Discussion of the interim conditions began with the 2017 HDR report titled *Mill Creek Watershed Alternatives Analysis and Optimization*. The purpose of that report was to model the collection system for existing, interim, and ultimate conditions. The findings were that the collection system is somewhat restricted at the existing conditions, and that restrictions are exacerbated as flows continue to increase due to watershed development. The modeled existing conditions were based on 2013 flow monitoring data. The modeling showed that a 10-year unrestricted peak storm event would result in 107 mgd to the MCR WWTP; however, 9 mgd of that total flow is from offsite collection system pump stations that do not go to the IPS. This results in a 10-year unrestricted peak storm flow to the IPS of approximately 98 mgd. In other words, once the collection system improvements are completed, the IPS will see a peak flow of 98 mgd. The dry weather firm pumping capacity is 24 mgd, which matches up to the maximum capacity of the mechanical plant. The mechanical plant UV Disinfection system is the limiting factor. To get more flow through the mechanical plant, significant improvements would have to be made to increase the pumping capacity and the UV Disinfection system capacity. Rather than recommending these improvements as part of the Interim Improvements, instead it is recommended to maintain a 24 mgd through the mechanical plant for treatment and add 35 mgd of wet weather pumping capacity to the IPS. Table 3-2 provides a summary of the IPS Interim Condition flows.

Table 3-2 IPS Interim Flow Summary

Dry Weather Pumping Station	Flow (MGD)
Current IPS Low Flow Condition (Diurnal Low)	4.75
Dry Weather Firm Pumping Capacity	24
Mechanical Plant Treatment Capacity	minus 24
Minimum Required Interim Capacity Upgrade	= 0
Wet Weather Pumping Station	**Flow (MGD)**
Current IPS Peak Unrestricted Flow	98
Dry Weather Firm Pumping Capacity	minus 24
Required Interim Wet Weather Capacity	= 74
Existing Firm Wet Weather Capacity	minus 39
Minimum Required Interim Capacity Upgrade	35 =

3.1.2 Interim IPS Recommended Improvements

Prior to this Facility Plan for the MCR WWTP, an extensive alternative analysis was done for the Tomahawk Creek (THC) WWTP Expansion. The results of this analysis can be used to inform the planning of the MCR Expansion. The THC WWTP is a good comparison because it is a similarly sized facility (19 mgd annual average (AA) flow), with similar wastewater characteristics, is owned and operated by JCW, and has actual market costs for treatment technologies provided by a Contractor. Part of the THC WWTP design included adding influent pump station capacity. The analysis for that project selected a wet-pit submersible type pump station. Since the MCR WWTP is similar to THC in many ways and the existing IPS at the MCR WWTP is a wet-pit submersible pump station, a similar approach was used at the MCR WWTP for the Interim Improvements. The benefits of this approach include ease of retrofit, the lowest capital cost approach, and JCW familiarity. It should be noted, however, that there are other types of pumps that could also provide benefit in this application. Other pump selections should be reviewed as part of the detailed design of the Interim Improvements project.

It will be difficult to add 35 mgd capacity to the Wet Weather Pumping Station by simply upsizing the pumps. The installed pumps are nearly 20 mgd each. To get enough firm capacity out of the existing pump station, each pump would have to be approximately 37 mgd because there are only three pumps installed. Additional concerns with simply upsizing the pumps are the wetwell hydraulic limitations, pump turndown ability, and medium voltage adjustable frequency drives (AFDs). The existing wetwell was laid out based on the dimensions of the existing pumps. Increasing the size of the pumps will likely make the wetwell non-compliant with Hydraulic Institute (HI) standards. A 37 mgd pump would likely not have enough turndown to operate at flow conditions that are just above the dry weather pumping capacity of 24 mgd. Lastly, a 37 mgd pump would be greater than 500 horsepower (hp), which gets into medium voltage AFDs. Medium voltage equipment adds cost and complexity, as well as a potential impact to service options.

A more feasible way of increasing the firm capacity of the IPS Wet Weather Pumping Station includes adding more pumps to the wetwell. If the size of the existing IPS pumps remained the same, it would take a total of five pumps installed to get enough firm capacity. That means two additional 17.5 mgd pumps would have to be added. Looking at the dimensions of the existing Wetwell No. 3, the pumps are evenly spaced along the eastern wall of the wetwell. There is not enough space to add even one additional similarly sized pump along this same lineup.

In the previously mentioned 2017 report by HDR, increasing the capacity of the Wet Weather Pumping Station was discussed in Section 8.2.2 and in Appendix D. The summary of the recommended improvement was to replace the 3 existing pumps with three (3) 24.7 mgd pumps, with an additional 24.7 mgd standby pump. HDR also recommended installing a new 24-inch force main for the new standby pump, along with a new metering vault and some miscellaneous electrical improvements. The new force main was proposed to tie into the existing Drop Box Structure, and 4 installed 24.7 mgd pumps would provide a firm capacity of 74 mgd. The only question would be how to orient this additional pump given the limited space on the east wall. HDR notes in the 2017 report that they worked with Flygt to develop the layout shown in Figure 3-2.

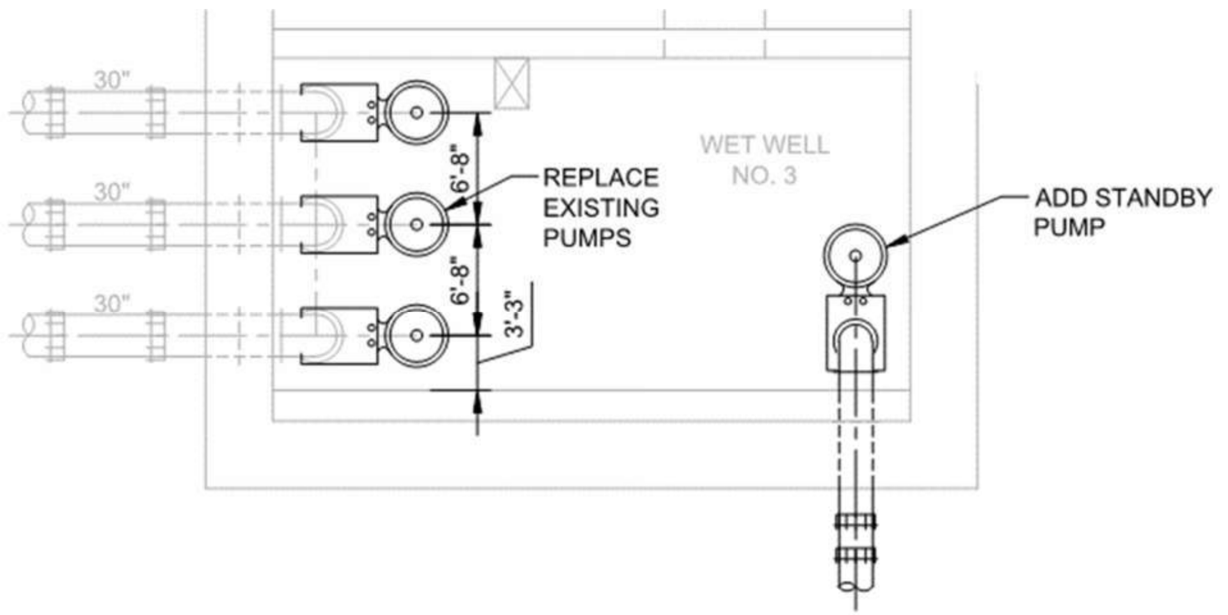

Figure 3-2 HDR Recommended Wet Weather Pumping Arrangement

Although Flygt has reviewed the layout presented in Figure 3-2, it should be noted that the dimensional layout and the flow patterns into Wetwell No. 3 do not meet the current HI guidelines. Since the capacity of the pump station will be increased by 40 percent, it is recommended that the wetwell layout be physically modeled to see if there are any undesirable flow patterns. There are three large labs who perform these tests in the U.S. – Clemson Engineering Hydraulics in South Carolina, Alden Labs in Massachusetts, and Northwest Hydraulic Consultants in Washington State. If physical modeling determines that the layout in Figure 3-2 is unacceptable, then another potential layout would be to have all four pumps in a line along the south wall. This layout would also require physical modeling since it would not meet HI guidelines. It would also require some rerouting of the discharge piping but may present some hydraulic advantages.

Another important consideration in the wet weather pump selection is the total dynamic head (TDH) of the selected pumps. The existing wet weather pumps send flow to the drop box structure, which then flows to the lagoon cells. The existing installed wet weather pumps are rated for a TDH of 70 feet. A hydraulic analysis of the existing system shows a recommended TDH of 85 feet. Based on this, it is thought that the existing pumps are slightly undersized. As discussed in TM 8 – Site Optimization and MOPO, during plant construction, wet weather will also be pumped to the drop box structure, then directly to Cell 8. The peak water surface elevation in Cell 8 is reduced when compared to the other lagoon cells. This results in an interim max TDH of 75 feet. Lastly, once the MCR WWTP Expansion is complete, the wet weather pumps will send flow to the Filter Complex for Auxiliary Treatment. Using the hydraulic profile presented in TM 8, an ultimate TDH of 78 feet can be calculated for the wet weather pumps. Therefore, if the wet weather replacement pumps are sized with a TDH to meet the existing TDH conditions, they will be sized adequately for all future conditions. Below, Table 3-3 provides a summary of the recommended wet weather pumps.

Table 3-3 Recommended Wet Weather Pumps

COMPONENT	COMPONENT SUMMARY
Wet Weather Pumps	Number of Pumps: 4 Pump Type: Submersible, non-clog Rated Capacity, gpm (each): 17,150 Head at Rated Capacity, ft: 78 Wet Weather Firm Pumping Capacity, mgd: 74.1 Motor, hp (each): 415 Drive Type: AFD

In addition to increasing the firm capacity of the Wet Weather Pumping Station, there are additional recommended improvements as part of an IPS Interim Improvements project. As previously mentioned, the existing bar screens are original to the IPS. The existing bar screen motors are not submersible and are shut off during very high flows. This results in poor screening during high flows, which increases the screen blinding and likely contributes to flooding within the IPS during peak flow events. It is recommended to replace the existing bar screens with mechanically cleaned chain and rake style bar screens. These types of screens will allow easy access to the drive unit and will allow for much more frequent cleaning of the bar screens than the existing climber style (or reciprocating rake) screens in this deep application.

As part of the screen replacement review, a hydraulic analysis of the channels was completed. The hydraulic analysis was focused on maintaining an approach velocity below three feet per second (fps). Since these screens will be in operation after the MCR WWTP Expansion, the hydraulic analysis was based on ultimate peak flows. In discussions with JCW plant staff, it was indicated that, at existing high flows, the water level in the influent chamber gets high enough to occasionally bypass the screens. As such, it is recommended to raise the height of the channels in order to minimize potential surcharging in the influent chamber at high flows. Other recommended IPS improvements include replacing the existing conveyors with a sluice trough and screenings compactor, and demolition of the installed exterior bridge crane. The installed bridge crane is not big enough to remove the existing installed wet weather pumps; however, instead of replacing the existing bridge crane with a larger one, it can be more economical to not install a crane and instead hire a company that can remove the pumps when they need servicing. A summary of the recommended equipment is provided in Table 3-4 and recommended IPS improvements are shown in Figure 3-3.

Table 3-4 Interim Improvements Screening Equipment

COMPONENT	COMPONENT SUMMARY
Coarse Screening	Type of Screen: Vertical, Mechanical, Front rake cleaned Number of Screens: 4 Channel Width, ft: 4 Channel Depth, ft: 60 Bar Screen Spacing, in: 1/4 Screen Inclination, deg: 80 Capacity, mgd (per screen): 42 Motor, hp (each): 5
Washer/Compactor	Number of Units: 1 Volume Capacity, cf/hr: 140 Motor, hp (each): 5

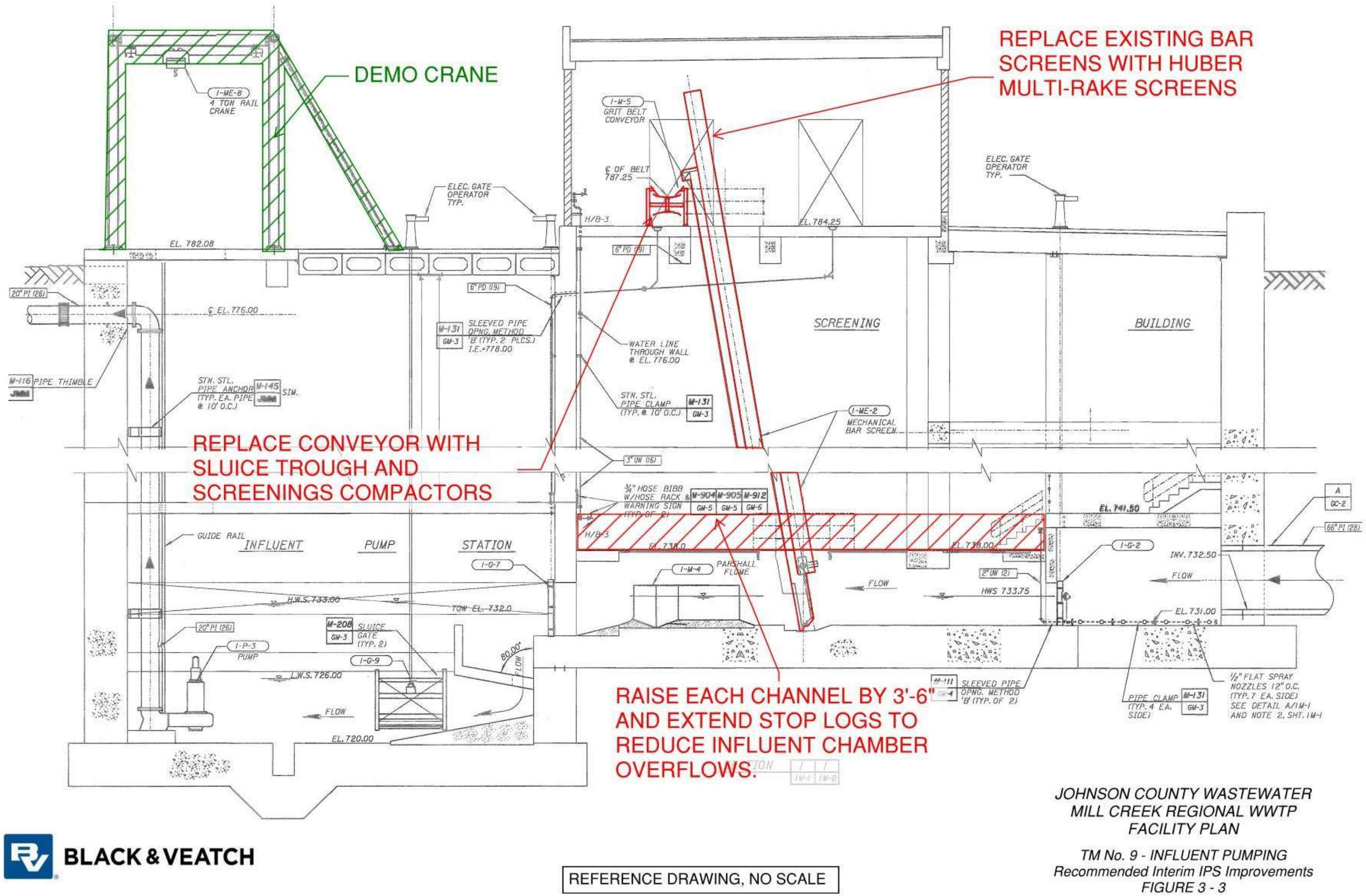

Figure 3-3 **Recommended Interim IPS Improvements**

As part of the IPS Interim Improvements project, it is recommended to build a stand-alone, single-story electrical building. The existing IPS electrical building is approximately 500 square feet (sf) and space is maxed out. For comparison purposes, the electrical room in the THC WWTP Peak Flow Pump Station is approximately 1,400 sf. The THC WWTP Peak Flow Pump Station is a good comparison to the IPS because the pumps are similarly sized and there is a comparable number installed. Therefore, a 1,400-sf electrical building is recommended. The new building will house all electrical equipment associated with the Interim Improvements, as well as all future IPS electrical equipment associated with the MCR WWTP Expansion. It is recommended the new electrical building be located to the south of the existing IPS electrical room. This is the recommended location because it is close to the existing electrical room and there are minimal crossings of in-service infrastructure. One of the challenges of building an electrical building in this area is the proximity to Mill Creek. Looking at the Johnson County Automated Information Mapping System (AIMS) in Figure 3-4, the extents of the existing 100-year floodplain around the IPS are shown in the darker blue. The predicted future 100-year floodplain extents are shown in the lighter blue. There is approximately 40-feet between the existing IPS and the future 100-year floodplain, which is adequate for the new electrical building. The existing electrical room floor at the IPS is at an elevation of 780.58-feet, which is about 1.5 feet above the 500-year floodplain. For additional protection of the new electrical building, it would be recommended to match the floor elevation of the existing electrical room.

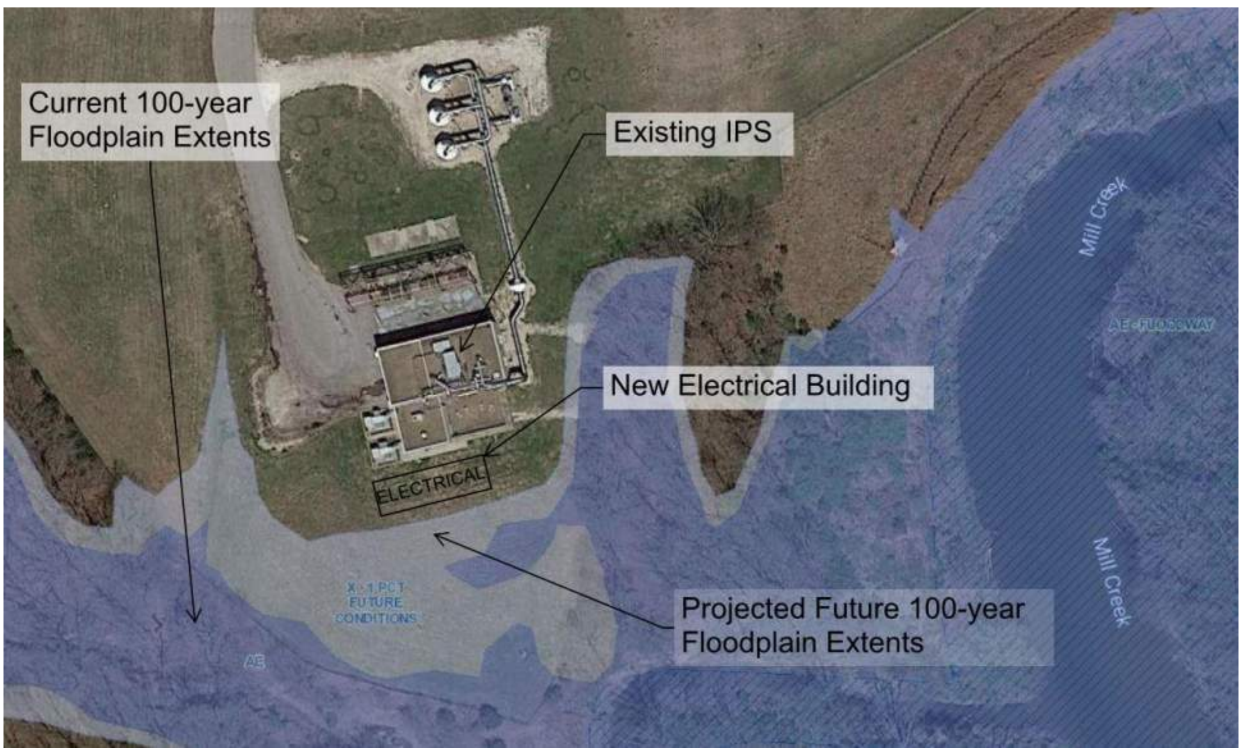

Figure 3-4 Interim Electrical Building Location

Lastly, it is important to discuss the existing effluent diffusers in the Kansas River. The effluent tunnel was installed as part of the Contract 10 work in 2010. The gravity discharge effluent tunnel connects to the Kansas River effluent diffuser pipe. The diffuser was designed to discharge up to 105 mgd through the 24-inch check valves. In previous reports it has been stated that upsizing to

36-inch check valves will increase the diffuser capacity to 132 mgd. As stated earlier in this TM, MCR WWTP can currently receive up to 108 mgd from the collection system. As such, the effluent hydraulics were reviewed again using an EPANet model. It was determined that flows up to 108 mgd can be conveyed using the existing check valves, as long as the level in the Kansas River doesn't exceed the 300-year flood elevation (772.50 ft). If flood conditions are beyond 300-year elevations and MCR WWTP was at 108 mgd, one potential layer of protection would be to utilize the interim outfall line and discharge directly to Mill Creek during these extreme events. Prior to the Interim Improvements, it is recommended to take a final review of the tunnel hydraulics to confirm the hydraulic capacity. Costs for upsizing the existing check valves are not included in the Interim or Ultimate improvements. It is believed this work will be a separate project.

The opinion of probable project cost (OPPC) for the IPS Interim Improvements project is presented in Table 3-5. The miscellaneous cost adders are included on top of the subtotal because it is expected that this project will take place outside of the MCR WWTP Expansion. Except for contingency, Engineering Legal and Administration (ELA) fees, and JCW Admin fees, the same percentages that were used in the MCR WWTP Expansion OPCC were used for this future project. Given the scale of the MCR WWTP Expansion project, 20 percent contingency is expected to be sufficient, however, this project is much smaller, so it is appropriate to use 30 percent contingency at a planning level. Similarly, given the scale, reduced ELA and JCW Admin fees were used for the MCR WWTP Expansion costs. For this OPPC, typical percentages have been used for ELA and JCW Admin fees. The OPPC is in January 2020 dollars.

Table 3-5 IPS Interim Improvements OPPC

IMPROVEMENT	COST
Pumping	$1,358,000
Screening	$2,090,000
Electrical Building	$500,000
Subtotal	**$3,948,000**
Sitework (20%), Electrical (20%), I&C (5%)	$1,777,000
Subtotal	**$5,725,000**
General Requirements (16%) & Contractor O&P (11%)	$1,546,000
Subtotal	**$7,271,000**
Contingency (30%)	$2,181,000
Opinion of Probable Construction Cost (OPCC)	**$9,452,000**
ELA (20%) and JCW Admin Fee (1.75%)	$2,056,000
Wetwell Pump Station Physical Modeling	$100,000
Opinion of Probable Project Cost (OPPC)	**$11,608,000**

- Costs are presented in January 2020 dollars
- OPCCs are at a conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high)

The pumping line item includes a vendor quote for four (4) 24.7 mgd pumps, demolition of the existing pumps, installation of the new pumps, and piping modifications. The screening line item includes costs for a vendor screen replacement quote, conveyance and compaction equipment, demolition and installation, miscellaneous channel improvements, and demolition of the bridge crane. The electrical building line item includes costs for a 1,400-sf single-story structure.

3.2 ULTIMATE IPS IMPROVEMENTS

The ultimate IPS improvements are recommended to be part of the MCR WWTP Expansion project. As such, for the purposes of this Section, it is assumed that the previously discussed IPS Interim Improvements have been implemented.

3.2.1 Ultimate Flow Summary

As discussed in previous TMs, the ultimate peak day flow at the MCR WWTP is 126 mgd. At peak conditions, half the flow (3Q) will be sent to Headworks and through secondary treatment. The other half of the peak day flow will be sent to the Filter Complex. As previously stated, the offsite collection system pumping stations force mains will be routed to the Headworks Building and, based on previous reports, it is estimated that the peak offsite pump station flow is 10 mgd. This means that the ultimate peak flow to the IPS is approximately 116 mgd.

One of the future NPDES permit requirements will be to maximize flow through the secondary treatment processes. In other words, flows cannot be sent directly to the Filter Complex until the MCR WWTP flow exceeds 63 mgd (3Q). If 10 mgd is being pumped to the Headworks Building from offsite pump stations, then the IPS Dry Weather Pumping Station needs to be able to pump a total of 53 mgd. In addition, as mentioned in TM 4 – Auxiliary Treatment, backwash from the Filter Complex will be sent back to the Headworks Building. At peak conditions, the filter backwash can be as much as 6 mgd. If 10 mgd was coming to Headworks from the offsite pump stations, and the Filter Complex was operating at peak conditions, 47 mgd would be required out of the Dry Weather Pumping Station.

Based on the previous paragraph, 47 mgd is the minimum capacity of the IPS Dry Weather Pumping Station, however, given the locations of the offsite pump stations, it is unlikely that the MCR WWTP and all the offsite collection system pump stations will be peaking at the same time. Because of this potential range of flows associated with the offsite collection system pump stations and the filter backwash, it is recommended that the Dry Weather Pumping Station is sized for a firm capacity of 63 mgd. That way a total of 63 mgd can always be sent through secondary treatment regardless of what is happening at the offsite collection system pump stations and the filter backwash.

If 63 mgd is what is needed out of the IPS Dry Weather Pumping Station, and the installed firm capacity is 24 mgd, then it can be calculated that an additional 39 mgd of IPS dry weather pumping capacity is required as part of the MCR WWTP Expansion. Table 3-6 presents a summary of how it was determined that an additional 39 mgd is needed from the IPS Dry Weather Pumping Station.

Table 3-6 IPS Ultimate Flow Summary

Dry Weather Pumping Station	Flow (MGD)
Ultimate Peak Secondary Flow (3Q)	63
Flow range from Offsite Pump Stations	minus (0 - 10 wide range of flows possible
Filter Backwash Peak Flow Range	0 – 6)
Required Dry Weather Pumping Capacity	= 47 - 63
Recommended Dry Weather Pumping Capacity	63
Interim Dry Weather PS Firm Capacity	minus 24
Required Ultimate Dry Weather Pumping Upgrade	39 =
Wet Weather Pumping Station	**Flow (MGD)**
Mill Creek Interceptor Ultimate Peak	116
Dry Weather Pumping Minimum	minus 47
Maximum Ultimate Wet Weather Pumping	69 =
Recommended Ultimate Wet Weather Pumping	minus 6 mgd, for provided dry weather pumping = 63
Interim WW Pumping Capacity	minus 74
Required Ultimate WW Pumping Upgrade	= 11 mgd excess

As presented in Table 3-6, if the ultimate Dry Weather Pumping Station is sized for a firm capacity of 63 mgd, then the ultimate Wet Weather Pumping Station also needs to be sized for a firm capacity of 63 mgd since the total peak flow is 126 mgd. After the IPS Interim Improvements, the installed Wet Weather Pumping Station capacity will be 74 mgd, creating an excess pumping capacity of 11 mgd. In other words, once the IPS Interim Improvements are completed, no additional wet weather pumping improvements are required.

3.2.2 Recommended MCR WWTP Expansion IPS Improvements

Similar to the wet weather pumping improvements discussed in Section 3.1.2, it would be challenging to achieve a firm dry weather pumping capacity of 63 mgd by simply upsizing the four installed pumps. Each pump would have to be approximately 21 mgd instead of the currently-installed 8 mgd. While there might be a 21 mgd pump that can meet the system head conditions, the concern is pump turndown ability. When the MCR WWTP Expansion is completed, it is estimated that the AA flow will be 12 mgd. If that is the case, the diurnal low flow condition is estimated to be close to 6 mgd. It is unrealistic to expect a pump to be able to routinely turn down approximately 70 percent. A more realistic scenario is to increase the number of installed pumps.

At a planning level, an approximate 50 percent turndown rate is a realistic assumption. If the diurnal low is estimated around 6 mgd, this would make the pumps approximately 12 mgd. To achieve a firm capacity of 63 mgd while minimizing turndown to 50 percent, it is recommended to install six 12.6 mgd pumps. As previously presented in Table 3-1, the existing dry weather pumps have a TDH of 84 feet. A preliminary hydraulic analysis of the new system concluded that the new dry weather pumps will have an increase in TDH. Although the length of piping will be reduced, the increased static head associated with pumping to the new Headworks Building will require each of the IPS dry weather pumps to have a slight increase in TDH. A summary of the recommended IPS dry weather pumps is presented below in Table 3-7.

Table 3-7 Recommended Dry Weather Pumps

COMPONENT	COMPONENT SUMMARY
Dry Weather Pumps	Number of Pumps: 6 Pump Type: Submersible, non-clog Rated Capacity, gpm (each): 8,750 Head at Rated Capacity, ft: 98 Dry Weather Firm Pumping Capacity, mgd: 63 Motor, hp (each): 385 Drive Type: AFD

Wetwell Nos. 1 and 2 face a similar challenge to Wetwell No. 3, given the current pump orientation there is not enough room to add additional pump slots. If the physical modeling for Wetwell No. 3 concludes that four pumps along the south wall is acceptable, then this same approach could be used for Wetwell Nos. 1 and 2. In Wetwell No. 2 the 2 existing 8 mgd pumps would be replaced with the larger pumps, then the 2 pumps in Wetwell No. 1 would be replaced with four 12.6 mgd pumps installed along the northwest wall. This orientation would require piping modifications; however the force mains will have to be rerouted anyway as they are going to new locations as part of the MCR WWTP Expansion.

In the case that four pumps along one wall is determined to not to be an acceptable layout, another alternative would be to build an additional dry weather wetwell as shown in Figure 3-5.

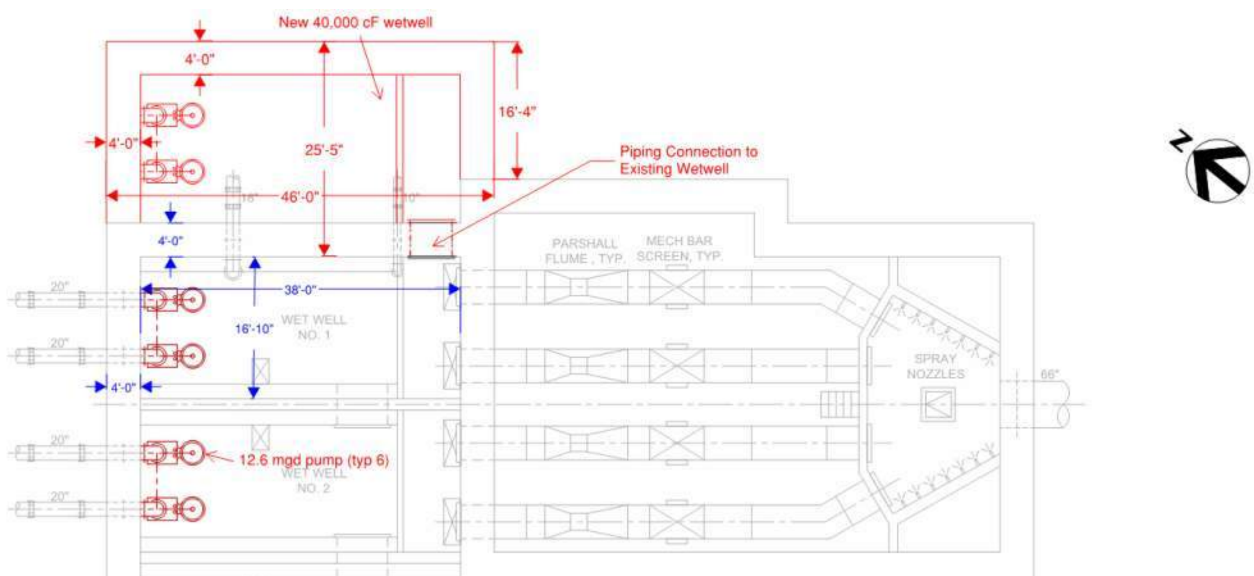

Figure 3-5 Recommended MCR WWTP Expansion IPS Pumping Improvements

As shown in Figure 3-5, the new wetwell would be the same volume as the existing wetwells and would tie in to the existing Wetwell No. 1 wall. The existing grade around the IPS is approximately at an elevation of 780 feet. The bottom of the IPS is approximately at an elevation of 720 feet. To be able to build a new common wall wetwell, excavation down to an elevation of at least 720 feet will be required. The IPS is nearby to the banks of Mill Creek, so an excavation of this size will be sure to encounter groundwater. When the IPS was originally constructed, a reinforced concrete pile shoring system was installed around the perimeter of the IPS. The shoring system was left in the ground once construction was completed. The piles begin at an approximate elevation of 20 feet below existing grade and terminate at bedrock. The function of this shoring system was to keep groundwater out of the excavation to the extent possible. To construct a new dry weather wetwell, it is anticipated that a similar type shoring system will need to be used. Figure 3-6 shows the extent of the existing shoring system, as well as the extent of a new shoring system for the wetwell addition.

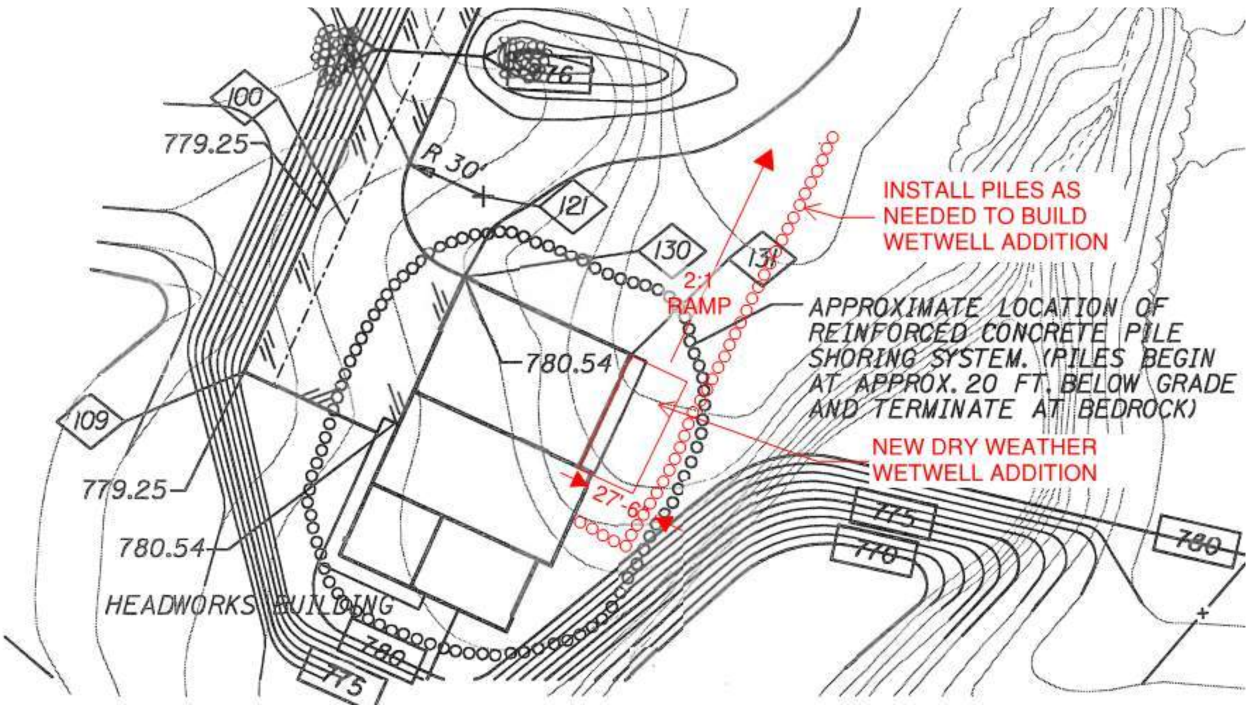

Figure 3-6 Existing and Recommended IPS Shoring

To the extent possible, the existing shoring system will continue to be used. Where the new piles are shown to intersect the existing piles, some of the previously installed piles may need to be replaced. Once all the piles are installed, excavation can begin. The footprint of the new wetwell will be excavated down to an elevation of 720 feet. Then a ramp will be sloped to the northeast until the elevation is back to the existing grade. When groundwater is encountered, dewatering will need to begin until the excavation area is dry. Eventually, the excavation will be deep enough that the new wetwell can be constructed. Once completed, this area will be backfilled and the piles will remain in the ground.

Each of the 3 Dry Weather Pumping Station Wetwells will have two 12.6 mgd pumps. The discharge piping will connect to a common header that sends flow to the Headworks Building. As part of the construction of the new wetwell, an opening between the new and existing wetwells will be constructed. The details of this opening will be determined during the detailed design phase; however, it is anticipated an isolation gate will be used to control the flow.

The OPCC for the MCR WWTP Expansion IPS Improvements is presented in

Table 3-8. It is important to note that the additional cost adders such as sitework, electrical, contingency, etc. are not included in this summary because they are included in the overall Facility Plan Costs.

Table 3-8 MCR WWTP Expansion IPS Improvements

IMPROVEMENT	COST
Sitework	$2,420,000
Structure	$1,120,000
Equipment	$1,380,000
Subtotal	**$4,920,000**

- Costs are presented in January 2020 dollars
- OPCCs are at a conceptual level (AACEI Class 4: -15% to -30% low, +20% to +50% high)

The sitework line item includes costs for excavation, fill, piles, and dewatering. The structure line item includes concrete for the new wetwell, labor, a new valve vault, and the connection to Wetwell No. 1. The equipment line item includes costs for demolition of the existing pumps, cost for new pumps, installation of the new pumps, and piping within the IPS. The piping replacement from the IPS to the Headworks Building and from the IPS to the Filter Complex are captured in the overall sitework cost adder.

4.0 Summary of Findings and Recommendations

4.1 COLLECTION SYSTEM OFFSITE PUMPING STATIONS

The MCR WWTP Expansion will require rerouting the existing offsite pump station force mains that are routed to the MCR WWTP. The existing force mains will be routed to the Headworks Building because the Flow Control Structure will be demolished. Cedar Mill Pump Station is to the northwest of the MCR WWTP and has its own dedicated force main. Rerouting this force main results in slightly less linear feet of pipe, but the static head is higher. This results in a slight increase in headloss, however, the headloss moves the system curve closer to the pump rated point, so no improvements are recommended at Cedar Mill Pump Station. Tooley Creek and 55th Street Pump Stations are both to the east of the MCR WWTP, and they arrive to the site in a single force main. The rerouting of the force main adds length and static head, which increases the overall system headloss. Both pump stations, however, were operating closer to the end of their curves, so adding headloss brings the pumps closer to the rated point. As such, no additional improvements are recommended at either pump station. It should be noted that all these three pump stations are older, so the pumps may start to reach the end of their useful lives, where they should be replaced with a similar pump.

4.2 MCR WWTP INFLUENT PUMPING

The MCR WWTP Expansion project is estimated to be completed by the end of 2035. Based on findings in previous reports, existing flows to the MCR WWTP exceed the installed pumping capacity at the IPS prior to the expansion. These peak flows will continue to increase over the next 15 years. As such, an IPS Interim Improvements project is recommended at the MCR WWTP. The recommended Interim Improvements project will increase the Wet Weather Pumping Station capacity to 74 mgd. It is recommended that the previously recommended Wetwell No. 3 layout is physically modeled to confirm there are no undesirable flow patterns since the layout does not meet the HI guidelines. If modeling determines the recommended layout is unacceptable, there are a few different potential layouts that might work. Otherwise, additional modifications may be required. Additional recommended improvements — such as bar screen replacements, conveyance and compaction improvements, channel improvements, demolition of the existing bridge crane, and a new stand-alone electrical building — should also be included as part of the IPS Interim Improvements project.

Assuming the IPS Interim Improvements are completed prior to design of the MCR WWTP Expansion, there are a few additional recommended improvements that will build on the work done in that project. After the wet weather pumping capacity is increased to 74 mgd as part of the Interim project, no additional wet weather improvements will be required, however, the Dry Weather Pumping Station firm capacity needs to be increased from 24 mgd to 63 mgd. If physical modeling confirms four pumps along one wall is acceptable, the existing IPS dry weather wetwells may be able to be modified without structural modifications. Otherwise, it is recommended to build an additional dry weather wetwell that is located on the same wall as Wetwell No. 1. This construction will involve a major excavation, piles, and dewatering. Once this new wetwell is constructed, it is recommended to demolish the four existing eight mgd dry weather pumps. These pumps should be replaced with two 12.6 mgd pumps in each dry weather wetwell. These 6 installed pumps will increase the firm dry weather pumping capacity to 63 mgd.

DRAFT

MILL CREEK REGIONAL FACILITY PLAN

Technical Memorandum 10
Implementation

JCW NO. MCR1-BV-17-12
BV PROJECT 403165

PREPARED FOR

JOHNSON COUNTY KANSAS
Wastewater

OCTOBER 9, 2020

BLACK & VEATCH

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Acronyms and Abbreviations

Abbreviation	Meaning
A	
AA	Annual Average
AADF	Average Annual Daily Flow
ADF	Average Daily Flow
AGS	Aerobic Granular Sludge
ANSI	American National Standards Institute
AUX	Auxiliary
B	
BV	Black & Veatch
BAF	Biological Aerated Filters
BFE	Base Flood Elevation
BFP	Belt Filter Press
BioMag	Biological Flocculation System from Siemens
Bio-P	Biological Phosphorous
BLDG	Building
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
BTU	British Thermal Unit
C	
C	Hazen-Williams Equation Roughness Coefficient
CA	Calcium
CBOD	Carbonaceous Biochemical Oxygen Demand
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CEPT	Chemically Enhanced Primary Treatment
cf	Cubic Feet
CFD	Computational Fluid Dynamics
CFH	Cubic Feet per Hour
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CFUs	Colony Forming Units
CHP	Combined Heat and Power
cm	Centimeters
CNG	Compressed Natural Gas
COD	Chemical Oxygen Demand
CSOs	Combined Sewer Overflows
CT	Concentration Time
CWA	Clean Water Act
D	
d	Day
DAF	Dissolved Air Flotation
DFM	Dry Weather Forcemain
DGC	Digester Gas Control Building
DIG	Digester
DISC	Disc Filters
DLSMB	Douglas L. Smith Middle Basin
DN	Down
DO	Dissolved Oxygen
DP	Dual Purpose
DS	Domestic Water Supply
dt	Dry Ton
DWF	Dry Weather Flow
DWS	Drinking Water Supply
E	
E. coli	Escherichia Coli
EA	Each
EFF	Effluent
EFHB	Excess Flow Holding Basin
EL	Elevation
ELA	Engineering, Legal, Administrative
ENR	Enhanced Nutrient Removal
ENR	Engineering News Record
EPA	Environmental Protection Agency
EQ	Equalization
F	
F/M	Food/Microorganism Ratio
FEMA	Federal Emergency Management Agency
ff	Flocculated and Filtered
$ffCBOD_5$	Flocculated Filtered Carbonaceous Biochemical Oxygen Demand
ffCOD	Flocculated Filtered Chemical Oxygen Demand
FFE	Furnitures, Fixtures, and Equipment

Abbreviation	Meaning
ffTKN	Flocculated Filtered Total Kjeldahl Nitrogen
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FL	Flow Line
floc	Flocculent
FM	Flow Meter
ft	Feet
FTE(s)	Full Time Equivalent(s)
G	
gal	Gallons
GGE	Gallons of Gas Equivalent
GMP	Guaranteed Maximum Price
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
gph	Gallons per Hour
gpm	Gallons per Minute
H	
HEX	Heat Exchanger
Hf	Friction Head
HI	Hydraulic Institute
HL	Head Loss
hp	Horsepower
hr	Hour
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation, Air Conditioning
HWE	Headworks Effluent
HWLA	High Water Level Alarm
Hypo	Sodium Hypochlorite
I	
I&C	Instrumentation and Controls
I/I	Inflow and Infiltration
IC	Internal Combustion
IFAS	Integrated Fixed-Film Activated Sludge
in	Inches
IND	Industrial
INF	Influent
IPS	Influent Pump Station
IW	Industrial Water Supply Use
J	

Abbreviation	Meaning
JCW	Johnson County Wastewater
K	
kcf	Thousand Cubic Feet
KCMO	Kansas City, Missouri
KDA-DWR	Kansas Department of Agriculture-Division of Water Resources
KDHE	Kansas Department of Health and Environment
KDWPT	Kansas Department of Wildlife, Parks and Tourism
kWh	Kilowatt-hour
L	
L	Length, Liter
lb	Pound
LDP	Land Disturbance Permit
LF	Linear Feet
LOX	Liquid Oxygen
LPON	Labile Particulate Organic Nitrogen
LPOP	Labile Particulate Organic Phosphorous
LS	Lump Sum
LWLA	Low Water Level Alarm
M	
MAD	Mesophilic Anaerobic Digestion
MBBR	Moving Bed Bioreactors
MBR	Membrane Bio-reactor
MCC	Motor Control Center
MCI	Mill Creek Interceptor
MCR	Mill Creek Regional
mg	Milligrams
Mg	Magnesium
MG	Million Gallons
mg/L	Milligrams per Liter
mgd	Million Gallons per Day
min	Minute, Minimum
mJ	Millijoules
MLE	Modified Ludzack-Ettinger
MLSS	Mixed Liquor Suspended Solids
MM	Maximum Month
mm	Millimeter
MMADF	Maximum Month Average Daily Flow

Abbreviation	Meaning
mmBtu	Million British Thermal Units
MOPO	Maintenance of Plant Operations
mpg	Miles per Gallon
MPN	Most Probable Number
N	
NACWA	National Association of Clean Water Agencies
NaOH	Sodium Hydroxide (Caustic)
NCAC	New Century Air Center
NDMA	N-Nitrosodimethylamine
NH3-N	Total Ammonia
NOI	Notice of Intent
NOx-N	Nitrate + Nitrite
NPDES	National Pollutant Discharge Elimination System
NPS	Nonpoint Source
NPV	Net Present Value
NTS	Not to Scale
O	
O&M	Operation and Maintenance
OMB	Office of Management and Budget
OPCC	Opinion of Probable Construction Cost
OPPC	Opinion of Probably Project Cost
Ortho-P	Orthophosphate
OUR	Oxygen Uptake Rate
P	
P	Phosphorous
PAOs	Phosphorous Accumulating Organisms
PC	Primary Clarifier
PD	Peak Day
PDF	Peak Daily Flow
PE	Primary Effluent
PEW	Plant Effluent Water
PFE	Primary Filtered Effluent
PFM	Peak Flow Forcemain
PHF	Peak Hour Flow
PLC	Programmable Logic Controller
PO_4-P	Orthophosphate Phosphorous
ppd	Pounds per Day

Abbreviation	Meaning
pph	Pounds per Hour
PPI	Producer Price Index
ppw	Pounds per Week
ppy	Pounds per Year
PS	Pump Station
psf	Pounds per Square Foot
psi	Pounds per Square Inch
PWWF	Peak Wet Weather Flow
Q	
Q	Flow
R	
RAS	Return Activated Sludge
rbCOD	Rapidly Biodegradable Chemical Oxygen Demand
RDT	Rotating Drum Thickener
RECIRC	Recirculation
RIN	Renewable Identification Number
RPM	Revolutions per Minute
R&R	Repair and Replacement
RWW	Raw Wastewater
S	
SBOD	Soluble Biochemical Oxygen Demand
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
sCOD	Soluble Chemical Oxygen Demand
SCR	Secondary Contact Recreation
Sec	Second, Secondary
SF	Square Foot
SG	Specific Gravity
SHPO	State Historic Preservation Office
SLR	Solids Loading Rate
SMP	Stormwater Management Program
SND	Simultaneous Nitrification/ Denitrification
SOR	Surface Overflow Rate
SOURs	Specific Oxygen Uptake Rates
SPS	Sludge Pump Station
SRT	Sludge Retention Time

Abbreviation	Meaning
SS	Suspended Solids
SSOs	Sanitary Sewer Overflows
SSS	Separate Sewer System
sTP (GF)	Soluble Total Phosphorous (Glass Fiber Filtrate)
SVI	Sludge Volume Index
SWD	Side Water Depth
SWPPP	Stormwater Pollution Prevention Plan
T	
TBL	Triple Bottom Line
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TCPS	Tooley Creek Pump Station
TDH	Total Dynamic Head
Temp	Temperature
TERT	Tertiary
TF	Trickling Filters
TFE	Tertiary Filter Effluent
THC	Tomahawk Creek
THM	Trihalomethanes
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TN	Total Nitrogen
TOC	Top of Concrete
TP	Total Phosphorous
TPS	Thickened Primary Solids
TS	Total Solids
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
TYP	Typical
U	
µg/L	micrograms per Liter
USACE	United States Army Corps of Engineers
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	Ultraviolet

Abbreviation	Meaning
UV LPHO	Ultraviolet Low Pressure, High Output
UV MPHO	Ultraviolet Medium Pressure, High Output
V	
VE	Value Engineering
VFA	Volatile Fatty Acids
VFAs	Volatile Fatty Acids (Speciated)
VFD	Variable Frequency Drive
VS	Volatile Solids
VSL	Volatile Solids Loading
VSr	Volatile Solids Reduction
VSS	Volatile Suspended Solids
W	
W	
WAS	Waste Activated Sludge
WASP	Water Quality Analysis Simulation Program
WBCR-A	Whole Body Contact Recreation – Category A
WBCR-B	Whole Body Contact Recreation – Category B
WET	Whole Effluent Toxicity
WFM	Wet Weather Forcemain
WK	Week
WS	Water Surface
WWTF	Wastewater Treatment Facility
WWTP	Wastewater Treatment Plant
Y	
YR	Year

1.0 Introduction

The purpose of this technical memorandum (TM) is to summarize the outstanding implementation components of the Mill Creek Regional (MCR) Wastewater Treatment Plant (WWTP) Expansion for the improvements described in previous TMs. This TM includes a discussion of the site utility requirements, permit requirements, implementation schedule, and the Opinion of Probable Project Cost (OPPC). The utility section includes a description of the existing facilities, design criteria, and proposed upgrades. The permitting section includes a description of local, federal, and state permit requirements. The schedule and OPPC sections include a description of the methodology and the proposed schedule and OPPC.

This TM is one in a series of technical memoranda that will be incorporated into a Facility Plan report summarizing the recommendations for the future expansion of the MCR WWTP. Individual treatment processes and facilities are outlined in previous TMs.

1.1 BACKGROUND

Prior to this Facility Plan for MCR WWTP, an extensive alternative analysis was done for the Tomahawk Creek (THC) WWTP Expansion. The results of this analysis can be used to inform the planning of the MCR WWTP Expansion. THC WWTP is a good comparison because it is a similarly sized facility (19 million gallons per day (mgd) annual average (AA) flow) with similar wastewater characteristics, is owned and operated by JCW, and has actual market costs for treatment technologies provided by a Contractor.

The future utility demand was calculated based on the recommended facilities presented in previous TMs and using THC WWTP as a point of comparison. The schedule was developed based on the selected technologies and the recommended construction phasing described in TM 8 – Site Optimization and Maintenance of Plant Operations (MOPO). The recommended OPPC and overall Operation and Maintenance (O&M) costs were compiled based on the selected treatment technologies outlined in previous TMs.

2.0 Site Utility Requirements

Black & Veatch coordinated with Evergy, WaterOne, and Kansas Gas Service to determine the current municipal power, water, and natural gas capacities in the MCR WWTP area and identify expansion options for the future. Future demand calculations were developed for each service based on the facilities proposed in previous TMs and using THC WWTP as a point of comparison.

2.1 ELECTRICAL SERVICE

The MCR WWTP receives power from two substations: Edwardsville and 53rd & Mund. Evergy estimates that in the last year, MCR had a peak power demand of approximately 3 MW. In addition to meeting this demand, both substations have an excess capacity of approximately 3.5 MW each, totaling to 7 MW of available capacity for future improvements.

After the MCR WWTP future expansion, it is estimated that the running load at the WWTP will increase from 3 MW to approximately 7.5 MW at the peak wet weather flow rate of 126 MGD. During the annual average dry weather flow of 21 MGD, the running load will be approximately 4.5 MW.

Evergy stated that the peak future running load of 7.5 MW could be provided from each substation today. The MCR WWTP is the primary power consumer in the area, and there are no expected projects in the future that would reduce the available capacity of the substations. Evergy agreed that if additional capacity is required, the substations could be expanded, and the expansion cost could either be borne by the County (expansion capacity is reserved for County's use) or borne by Evergy (Evergy uses the expansion capacity as needed). There is no expectation of a future substation at the MCR WWTP by Evergy, but JCW should check in with Evergy every couple of years to monitor Evergy's electrical service conditions. The future power demand at MCR and the future power availability at the two substations should be confirmed during preliminary design.

2.2 MUNICIPAL WATER SERVICE

The service area where the MCR WWTP is located is served by one (1) 12-inch and one (1) 3-inch main that converge to one (1) 3-inch line at W 47th Street and Woodland Drive. The plant is serviced by one 2-inch domestic compound water meter and one 4-inch fire line connection. The 2-inch domestic water meter can handle a capacity of 160 gpm. A fire hydrant flow test was conducted on June 19, 2020 that measured a hydrant flow of 1,661 gpm with a pressure drop from 124 to 108 psi. This equates to a fire suppression flow of approximately 4,563 gpm at 20 psi. The current municipal water capacity at the MCR WWTP is displayed in Table 2-1.

Table 2-1 Current Municipal Water Capacity

	CAPACITY (GPM)
Potable & Service Water	160
Fire Suppression @ 108 psi	1,661
Fire Suppression @ 20 psi	4,563

After the expansion, it is estimated that the peak potable and service water demand at MCR will increase to 800 gpm. This demand was estimated based on the facilities selected for implementation at the MCR WWTP, using the THC WWTP as a point of comparison. In addition, the plant will have a fire suppression demand of approximately 1,250 gpm based on the Administration Building. The future peak water demand is shown in Table 2-2below.

Table 2-2 Future Peak Water Demand

	WATER DEMAND (GPM)
Potable & Service Water	800
Fire Suppression	1,250

Between the existing forcemains, there is ample capacity in the system to meet the demand. It is not anticipated that there will be any large developments in the area that will impact future demand. However, the plant's domestic water meter capacity will need to be increased to meet the future demand.

Adding a 4-inch domestic compound meter would cost approximately $150,000 and would add 600 gpm of capacity and a 6-inch domestic compound meter would cost approximately $350,000 and would add 13,000 gpm capacity. It is possible that the fire connection will need to be upsized as well, but this would incur a minimal cost since fire connections do not require meters.

To meet the calculated potable and service water demand, two four-inch meters or one six-inch meter would need to be installed. A third option is to keep the existing two-inch and install a four-inch meter. In this case, the capacity of the system would be 760 gpm, so the overall water demand would need to be reduced by 40 gpm. The municipal water demand should be evaluated in greater detail during preliminary design to determine the actual peak demand and identify appropriate system modifications. Additionally, a new fire flow test should be performed after WaterOne's new elevated tank near the MCR WWTP is brought online. The new tank will reduce the static pressure of the water service at the plant, but sufficient flow should still be available per WaterOne.

2.3 NATURAL GAS SERVICE

There is no natural gas service currently at the MCR WWTP. To reduce annual O&M costs, natural gas is recommended for heating at buildings. Natural gas is also recommended as a back-up heating system for the digesters.

After the MCR WWTP Expansion project, a gas demand of 26,000 cubic feet per hour (CFH) is estimated. This demand was estimated based on the facilities selected for implementation at the MCR WWTP, using the THC WWTP as a point of comparison. To meet the demand, a new gas service main will need to be installed. This service line would be an extension from an existing gas main and would include a gas meter. Kansas Gas Service was contacted about this and indicated a high-level cost estimate to install a service line extension. The high-level cost estimate was based on an approximate route that Kansas Gas Service determined which was not shared and therefore not included in this TM. The cost would include a new service fee and was estimated to be between $500,000 and $750,000.

The actual cost will be determined when the building site plan is finalized, and pressure requirements are known. The cost of the natural gas service connection is included in the total utilities cost. This is shown as a line item in the OPPC displayed in Table 5-3.

3.0 Construction Permit Requirements

The MCR WWTP is located in the City of Shawnee, Kansas. The MCR WWTP is bounded on the South and Southeast by Mill Creek, on the North and Northeast by W 47th Street, and on the West by City-owned recreational fields. All work will occur within the existing property. A small portion of the property on the south lies within the FEMA-regulated floodway. The floodway, unlike the surrounding floodplain fringe, is the area in which any obstruction will impact the 100-year flood elevation. No critical infrastructure will be developed in the floodway. Developments in the floodplain are allowed but must meet all City development ordinances. None of the proposed structures fall within the floodplain. However, the new plant access road to the east from Wilder Road will be in the floodplain.

Multiple permits from local, state, and federal agencies are anticipated as described below.

3.1 PERMITS FROM THE CITY OF SHAWNEE

- The City of Shawnee requires Building Permits, including a Site Plan, Land Disturbance Permit (LDP), and Floodplain Development Permit. The Floodplain Development Permit is only required for construction within the floodplain. The building permit will entail a detailed review of compliance with the building codes department. The site plan must address the suitable arrangement of structures, lighting, landscaping, site drainage; the promotion of public safety and convenience; and protection of surrounding property values. The building permit should be approved at the end of final design, just before bidding; however, coordination with the City of Shawnee Building Codes Department should begin at the initiation of detailed design.
- A Public Improvement Permit (PIP) and Stormwater Management Permit (SMP) are also required by the City of Shawnee. The PIP addresses the construction of a Stormwater Detention Basin. The SMP addresses the construction of any artificial watercourse to direct natural surface water or drainage from paved surfaces, structures, roads or improvements directly or indirectly. The PIP and SMP should be submitted near the end of preliminary design.
- Right-of-Way and Temporary Sign Permits are also required and should be obtained by the Contractor prior to construction.
- In addition to obtaining permits, JCW will need to coordinate construction truck haul routes with the City of Shawnee.

3.2 PERMITS AT THE FEDERAL & STATE LEVEL

- The United States Army Corps of Engineers (USACE) regulates Waters of the United States under Section 404 of the Clean Water Act. Section 404 permits (nationwide and individual permits) are required for impacts to wetlands and streams. A wetland delineation must be performed to identify wetlands and streams that may be impacted. In addition, work requiring a federal permit such as a USACE Section 404 permit requires coordination with the U.S. Fish and Wildlife Service with respect to threatened and endangered species. USACE involvement also requires coordination with the Kansas State Historical Society - State Historic Preservation Office (SHPO) and the Kansas Department of Wildlife, Parks and Tourism (KDWPT) with respect to clearance from these agencies. The USACE application process will take about six months, and the permits should be obtained a few months before construction so that construction falls within the five-year window of the permit term.

- Kansas Department of Health and Environment (KDHE) Notice of Intent (NOI) and general NPDES stormwater permit, which deals with erosion control and includes a Stormwater Pollution Prevention Plan (SWPPP). The permit and NOI application should be submitted during final design.
- Kansas Department of Agriculture – Division of Water Resources (KDA-DWR), which regulates construction of drainage structures and placement of fill in floodplains. The DWR floodplain and stream permit application should be submitted during final design.

3.3 OTHER COMMON PERMITS

The permits listed below are common permits that are not anticipated for this project.

- Preliminary and Final Plats are required by the City of Shawnee when land is divided into more than one tract, lot, or parcel.
- A groundwater pumping permit is required by the KDA-DWR for structures that permanently pump groundwater. Since this is not anticipated at the MCR WWTP, this permit will not be required.

At the THC WWTP, JCW covered the cost of all permits except for the right-of-way, temporary sign, and building permits that were required by the City of Leawood. These permits were paid for by the Contractor. It is anticipated that the right-of-way, temporary sign, and building permits will be paid for by the Contractor for the MCR WWTP Expansion as well.

4.0 Implementation Schedule

An implementation schedule was developed that spans from the initial design engineer selection until final demolition and closeout. The schedule includes construction manager at-risk (CMAR) services but would also be appropriate for traditional design-bid-build delivery. The schedule was developed so that the expanded plant will be online by year 2035, as required by the Integrated Management Plan. The schedule was separated into four phases: engineering design, CMAR pre-construction, construction, and permitting. The construction phase activities are based on the construction phasing alternatives recommended in TM 8 – Site Optimization and MOPO.

4.1 DESIGN AND CMAR PRE-CONSTRUCTION

After the engineer is selected, preliminary design will commence. At the same time, the CMAR selection process is recommended to begin. The CMAR should be selected early in the design to allow early development of the guaranteed maximum price (GMP) cost model, which will minimize risk and provide earlier cost certainty. The cost model will be updated throughout design and can be finalized at 95 percent design if needed for schedule considerations. After the design is complete and the GMP is established, the contract will be awarded, and the contractor will receive notice-to-proceed (NTP) for construction.

4.2 SITE FILL AND MOPO

After NTP, Site Fill and MOPO will begin. Piping will be installed as needed to allow Cell 6 to be used for WAS storage and Cell 8 to be used for wet weather treatment. When this is complete, Cells 3, 4, and 5 will be available for construction. Cells 3, 4, and 5 will be drained, sludge removed, and then filled with off-site material as recommended by a geotechnical engineer to complete the pre-loading phase. Once complete, the excess soil will be stored on-site and used as backfill during construction. The Site Fill and MOPO phase will last approximately one year.

4.3 CONSTRUCTION

The Filter Complex and UV Facility will be located on the south side of the plant, away from the lagoons. Since the construction of these facilities does not interfere with Site Fill and MOPO, this activity can begin after NTP and will last approximately one and a half years. Once the Filter Complex and UV Facility are online, Cell 8 will no longer be needed for wet weather treatment and can be drained and filled. The Plant Effluent Water (PEW) Pump Station is also planned for this area and can be constructed concurrently, although it is not a driver for Cell 8 decommissioning.

After Site Fill and MOPO is complete, construction of the remaining facilities can commence and will last approximately three years through substantial completion. As facilities are completed, they will go through startup and commissioning. It is estimated that the substantial completion and startup activities will begin about nine months before construction is complete and will last 18 months. The end of this activity is marked by substantial completion in December 2034.

4.4 SITE DEMOLITION

After substantial completion, the demolition and closeout phase will commence. This phase will consist of demolishing the existing mechanical plant and filling Cell 6. The completely-mixed cells (Cells 1 and 2) will be partially filled and reconstructed to convert them into a detention basin. Final site grading will be completed along with construction of any remaining support facilities and plant roads. In total, demolition and closeout will last approximately nine months. Refer to TM 8 – Site Optimization and MOPO for additional information on the phases of construction.

4.5 SCHEDULE SUMMARY

To have the MCR WWTP Expansion project completed and online by the year 2035, it is recommended that an engineer is selected and begins the design phase by March 2028. Engineering design will last approximately two years and eight months and will occur concurrently with CMAR pre-construction activities. The construction phase will begin after CMAR pre-construction activities are complete and will last approximately four years and nine months. The full implementation schedule is shown in Figure 4-1.

ID	Task Name	Start	Finish
1	**Engineer Selection**	**8/27/2027**	**3/27/2028**
2	**Engineering Design**	**3/28/2028**	**11/29/2030**
3	Preliminary Design	3/28/2028	12/27/2028
4	95% Detailed Design	12/28/2028	7/29/2030
5	100% Detailed Design	7/30/2030	11/29/2030
6	**CMAR Selection**	**3/28/2028**	**10/27/2028**
7	**CMAR Pre-Construction**	**10/28/2028**	**11/30/2030**
8	Develop GMP	10/28/2028	7/29/2030
9	Finalize GMP	7/30/2030	10/14/2030
10	Contract Award/Notice to Proceed	10/15/2030	11/30/2030
11	**Construction Permits**	**6/28/2028**	**11/29/2030**
12	PIP/Stormwater Management Permit (Shawnee)	6/28/2028	12/27/2028
13	Building Permits/Land Disturbance Permit (Shawnee)	12/28/2028	11/29/2030
14	NOI/NPDES Stormwater Permit (KDHE)	1/30/2030	7/30/2030
15	Floodplain and Stream Permit (KDA-DWR)	1/30/2030	7/30/2030
16	Section 404 Permits (USACE)	3/28/2030	9/23/2030
17	**Construction Phase**	**12/1/2030**	**9/1/2035**
18	Activity 1 - Site Fill & MOPO	12/1/2030	12/1/2031
19	Activity 2 - Filter Complex & UV Facility	12/1/2030	5/31/2032
20	Activity 3 - Construction	12/2/2031	6/1/2034
21	Activity 4 - Startup & Substantial Completion	6/1/2033	12/1/2034
22	Activity 5 - Demolition & Closeout	12/2/2034	9/1/2035

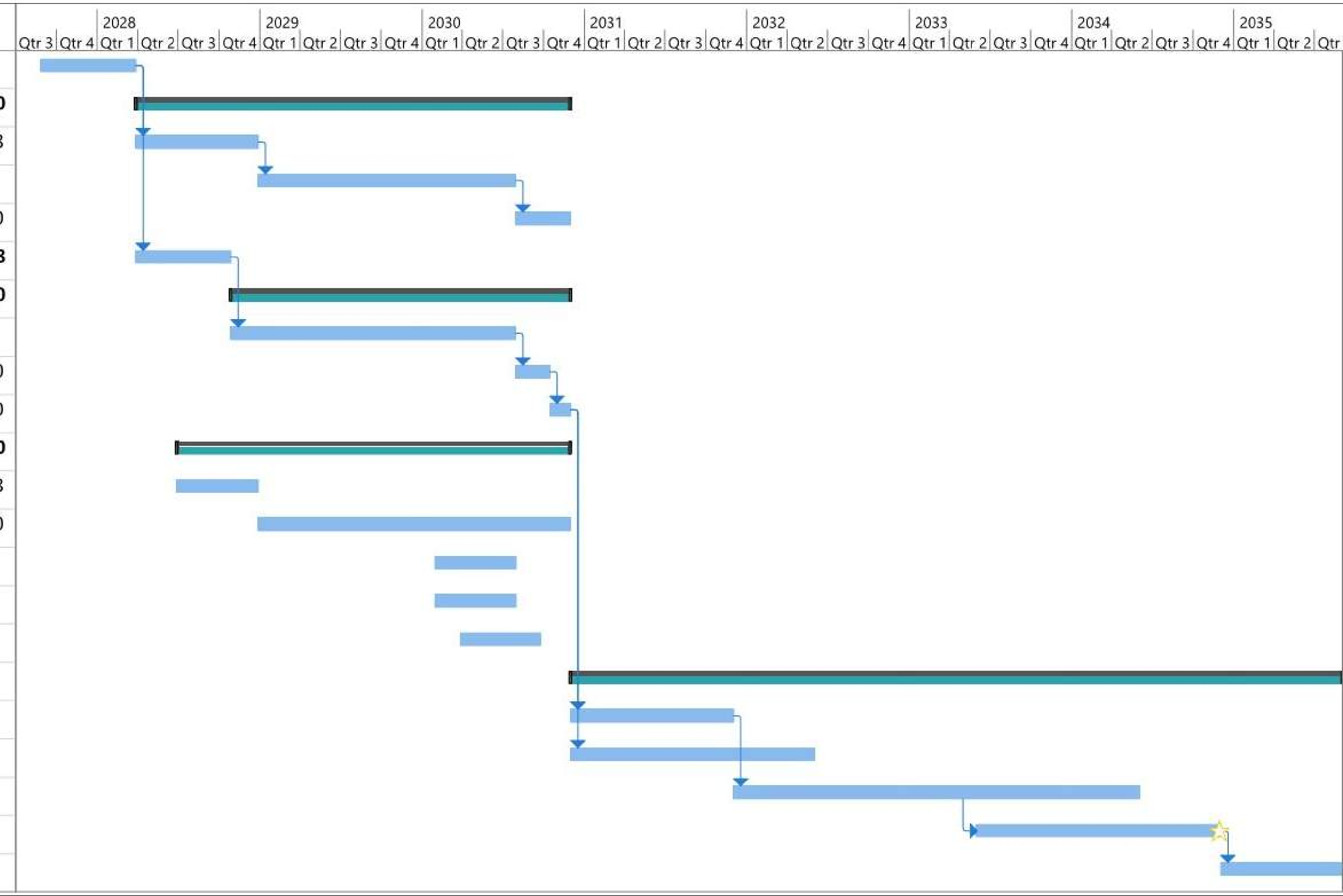

Figure 4-1 Implementation Schedule

5.0 Summary of Project Capital Costs

This TM includes an OPPC for all selected treatment processes in previous TMs. Project costs are also provided for interim Influent Pump Station (IPS) improvements. The phased construction of the IPS is described in TM 9 – Influent Pumping.

A list of potential plant expansion project value engineering (VE) savings is also presented. The value engineering options were developed by selecting alternative technologies, combining structures, and removing some facilities. Aside from implementing value engineering options, JCW could pursue a phased construction process if population growth is low. This would result in a higher overall project cost but would save money on the first phase of construction.

5.1 INTERIM INFLUENT PUMP STATION IMPROVEMENTS COSTS

Before the MCR WWTP Expansion, improvements will need to be made to the IPS to increase the wet weather capacity. In addition, the existing IPS equipment such as the bar screens and the belt conveyor are nearing the end of their useful life and will need to be replaced. The current condition of the pump station and proposed modifications are described in greater detail in TM 9 – Influent Pumping. The estimated costs of these interim improvements are displayed in Table 5-1. All costs are presented in 2020 dollars.

Table 5-1 Interim Influent Pump Station Improvements Capital Cost

IMPROVEMENT	CAPITAL COST
Pumping	$1,390,000
Screening	$2,090,000
Electrical Building	$500,000
SUBTOTAL	**$3,980,000**
Sitework – 20%	$796,000
Electrical – 20%	$796,000
I&C – 5%	$199,000
SUBTOTAL	**$5,771,000**
General Requirements – 16%	$923,000
Contractor O&P – 11%	$635,000
Contingency – 30%	$1,731,000
OPCC	**$9,060,000**
ELA – 20%	$1,812,000
Administration Fee – 1.75%	$159,000
TOTAL OPPC	**$11,031,000**

5.2 EXPANSION PROJECT COSTS

The capital costs associated with each structure are described in their respective TMs. When costs were scaled, the Engineering News Record (ENR) Building Cost Index (BCI) was used. Except for the future OPPC, all costs are presented in January 2020 dollars.

The capital costs developed for the facilities did not include excavation, backfill or any special foundation considerations. This was left out of the line items for each facility because the foundation requirements were not known at the time many of the individual facility costs were developed. In addition, THC WWTP construction costs were used as the cost basis for many facilities and THC facilities were constructed on a combination of drilled piers, rock, mat, and spread footings. All MCR WWTP facilities are expected to be constructed on mat and spread foundations with the exception of the Influent Pump Station Expansion, which will likely involve a deep foundation. The excavation and backfill line item cost presented in the MCR WWTP OPPC development is needed to account for these costs, as well as the construction multipliers applied to the correct costs.

Table 5-2 below presents a summary of the facility line item costs. All facility costs are presented in January 2020 dollars.

Table 5-2 Recommended Facilities Subtotal

FACILITY	CAPITAL COST
Influent Pump Station Improvements[1]	$4,920,000
Headworks Building	$8,847,000
Primary Sludge Pump Station	$1,704,000
Primary Clarifiers	$8,734,000
BNR Basin	$20,254,000
Basin Blower Building	$4,165,000
Final Clarifiers	$7,757,000
Final Sludge Pumping Station	$6,304,000
Sidestream Deammonification Building	$4,911,000
Filter Building	$9,251,000
UV Building	$6,121,000
PEW Pump Station	$1,038,000
Gravity Thickeners/Fermenters	$2,699,000
WASSTRIP Tank	$668,000
DAFs (1st & 2nd Stage)	$6,158,000
Thickening Building	$4,778,000
Dewatering Building	$10,933,000
Digesters	$12,603,000
Digester Control Building	$10,700,000
Phosphorus Recovery Building	$3,391,000
CNG Processing	$2,912,000
Septage Receiving	$867,000
Jet-Vac Truck Dumping	$303,000
Odor Control	$6,198,000
Administration and Maintenance Buildings	$5,406,000
Site Demolition	$1,500,000
Excavation / Backfill for Structures	$8,000,000
FACILITIES SUBTOTAL	**$161,122,000**

Notes:
[1] This cost does not include interim pumping improvements.

To get the complete project costs, several multipliers were used. Each multiplier was applied to the preceding subtotal. Multipliers were used to estimate Sitework, Electrical, and I&C improvements, along with other project cost components such as profit, fees, and contingencies.

One cost applied below the facilities subtotal is the site fill costs. This cost is for the required fill to pre-load Cells 3, 4, and 5 prior to the bulk of construction. The capital cost was estimated based on the volume of fill and was included as a separate line item so that the Sitework, Electrical, and I&C multipliers would not be applied to it.

Utility costs are combined as a line item with Furniture, Fixtures, and Equipment (FFE). Utilities include potable water, natural gas, and power. The capital cost associated with utilities accounts for water and natural gas costs are described in Section 2.0 above.

Whereas the OPPC is the total project cost, the Opinion of Probable Construction Cost (OPCC) is the amount that will be paid to the Contractor. The multipliers used in the OPCC/Projected CMAR GMP were based on the THC WWTP GMP, with the exception of the Contingency multiplier. This multiplier was increased to 20 percent from 9% for combined allowances and contingencies used in the THC WWTP GMP completed at 95% design. The increase is due to the preliminary nature of this cost opinion. At this level of design, the total construction contingency is typically 25 to 30 percent. A lower construction contingency was used for this cost opinion because the recent, complete CMAR costs available from THC were used as the basis for most of the MCR WWTP facility costs.

The final OPPC was presented in 2020 and 2031 dollars. Year 2031 was selected because the implementation schedule shows construction NTP at the end of 2030 and the base OPPC costs are based on January 2020 dollars. The long-term historical average rate of inflation in the United States is approximately three percent, thus, a three percent inflation rate was used for the future cost projection. For simplicity, the engineering cost was escalated to 2031 as well, although the midpoint of engineering design is approximately 18 months before the beginning of construction. The OPCC/Projected CMAR GMP and OPPC are presented in Table 5-3.

Table 5-3 Opinion of Probable Project Costs (OPPC)

FACILITY	CAPITAL COST[1]
FACILITIES SUBTOTAL	**$161,122,000**
Site Fill	$15,000,000
Sitework – 20%	$32,224,000
Electrical – 20%	$32,224,000
I&C – 5%	$8,056,000
SUBTOTAL	**$248,626,000**
General Requirements – 16%	$39,780,000
Contractor's Overhead and Profit – 11%	$27,349,000
SUBTOTAL	**$315,755,000**
Contingency – 20%	$63,151,000
Opinion of Probable Construction Cost / Projected CMAR Guaranteed Maximum Price (GMP)	**$378,900,000**
ELA – 18%	$68,202,000
JCW Administration Fee – 1.5%	$5,684,000
CMAR Pre-Construction Fee	$3,500,000
FFE, Utilities	$3,000,000
Opinion of Probable Project Cost (2020)	**$459,000,000**
Opinion of Probable Project Cost (2031)[2]	**$635,000,000**

(1) Costs presented are in January 2020 dollars except for the future 2031 cost.
(2) Future 2031 costs were escalated using 3% per year inflation.

The GMP will not be paid to the Contractor as a lump sum but will be paid on a monthly basis. Using the THC WWTP as the basis, monthly payments were estimated for the MCR WWTP from the beginning of engineering design through the end of construction. Figure 5-1 shows the monthly payment amounts along with the cumulative total paid.

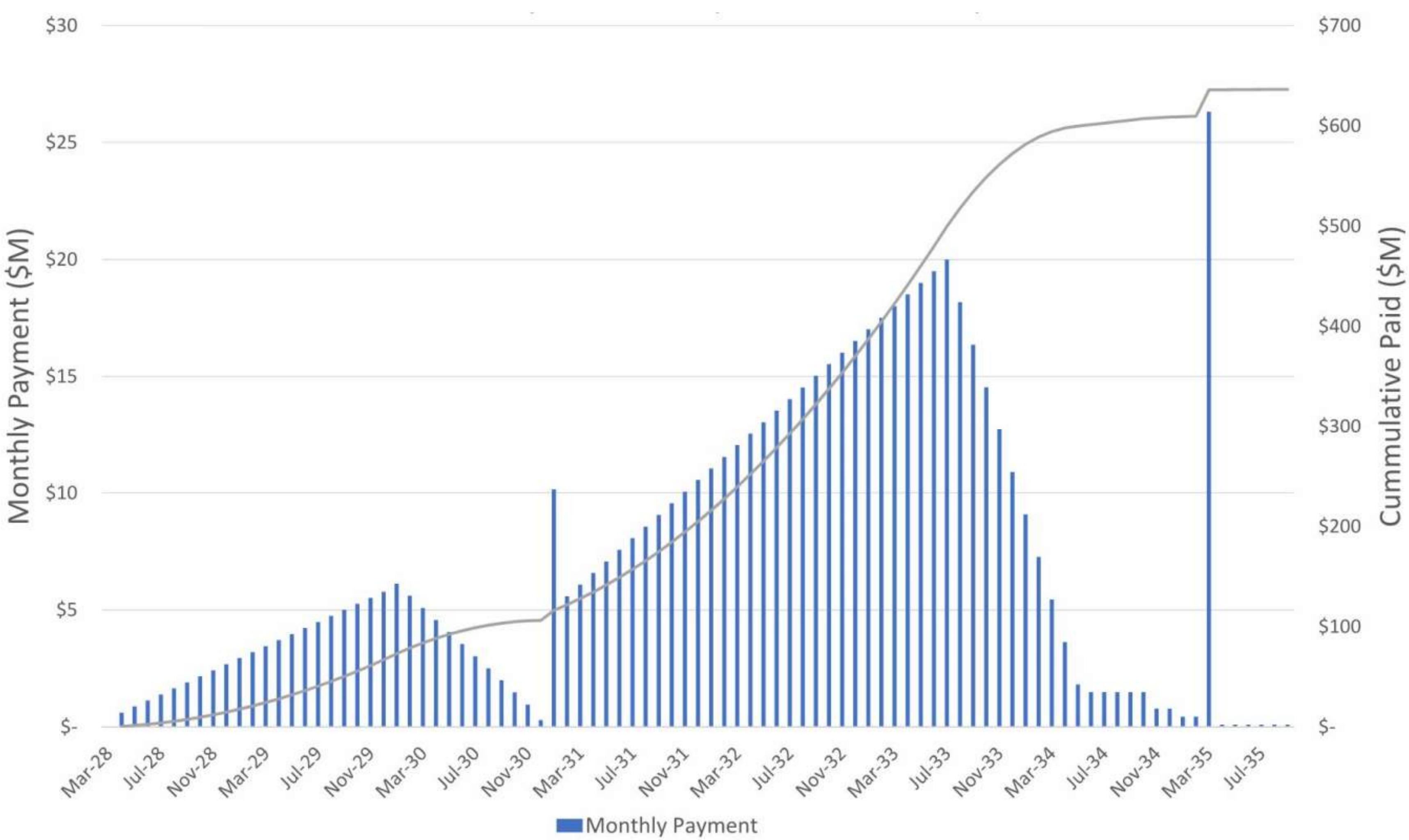

Figure 5-1 Projected Monthly Payments and Cumulative Paid Amount

5.3 EXPANSION PROJECT VALUE ENGINEERING OPTIONS

The OPPC presented in the previous section includes all of JCW's preferred treatment processes arranged for maximum flexibility and represent the best conservative estimate for planning purposes. There are options that could be pursued to lower project costs if that is needed considering JCW's Integrated Management Plan goals. This section includes eight major options that are feasible and that would still enable JCW to reliably meet permit limits while providing a highly operable and flexible plant. A table is provided at the end of this section summarizing the options described below. These value engineering options should be re-evaluated closer to implementation based on future market costs and JCW's preferences.

5.3.1 Eliminate Phosphorus Recovery

This phosphorus recovery process provides process benefits as well as a small revenue source from the end product that can be sold as a fertilizer. Process benefits include preventing struvite buildup, preventing excess nutrients from returning to the head of the plant, and producing higher quality biosolids at a lower volume; however, NPDES permit requirements could be met without the phosphorus recovery process. WASSTRIP was included with the main Ostara Fx process to provide the most phosphorus capture. Two stages of thickening are required to implement the WASSTRIP process. Capital cost savings represent the elimination of the Phosphorus Recovery Building, WASSTRIP and second stage DAFs.

5.3.2 Eliminate CNG Facility

The CNG Facility provides a means to recover and reuse the gas produced by the digesters as vehicle fuel. However, this facility does not affect the process of the plant and, thus, has no impact on the NPDES permit. Capital cost savings represent the elimination of the CNG Facility.

5.3.3 Eliminate Plant Effluent Water Pump Station

The PEW PS redirects treated effluent to multiple facilities on-site that would normally have a large service water demand. It is estimated that the average PEW demand will be roughly one quarter of the peak demand. Using this projection, the daily average PEW demand would be about 370 gpm. Depending on the future municipal water rate, the payback period may be a few years. Capital cost savings represent the elimination of the PEW PS.

5.3.4 Select Primary Clarifiers with Chemically Enhanced Primary Treatment

In TM 2 – Preliminary and Primary Treatment, traditional primary clarifiers (PCs) were recommended as the preferred primary treatment technology. If PCs with chemically enhanced primary treatment (CEPT) were implemented, significantly less surface area would be required for treatment, resulting in three (3) 105-foot-diameter clarifiers rather than four (4) 115-foot-diameter ones. There are additional capital and O&M costs associated with the chemicals used for CEPT during wet weather events. These costs are related to the use of chemical and the biosolids consequently produced but are minor compared to the savings resulting from the smaller footprint. Additionally, one primary sludge pump can be eliminated with one less clarifier constructed.

5.3.5 Select Disk Filters / Eliminate Primary Sludge Pump Station

As an alternative to traditional PCs or PCs with CEPT, Disk Filters could be implemented as the primary treatment technology. Disk Filters are a more expensive technology than traditional PCs, but implementing Disk Filters would have significant downstream process impacts that would result in overall cost savings. In addition, if Disk Filters were implemented, there would be no need for a separate primary sludge pump station. Capital cost savings represent the difference between traditional PCs and Disk Filters, elimination of the Primary Sludge Pump Station (PSPS), and cost savings from downstream process impacts. A more descriptive cost analysis between the alternatives is provided in TM 2 – Preliminary and Primary Treatment.

5.3.6 Combine Headworks Building and Primary Sludge Pump Station

If traditional PCs or PCs with CEPT are selected as the primary treatment technology, a PSPS will be required. Costs can be reduced by combining the PSPS and Headworks Building into a single facility. This can be accomplished by housing the primary sludge pumps in the basement of the Headworks Building, similar to the approach at the THC WWTP. Combing the structures increases the complexity of the building and may impact the critical path during construction. Combining the structures would also make it more difficult to remove equipment. Cost savings represent the elimination of the PSPS and housing the primary sludge pumping equipment in the Headworks Building.

5.3.7 Select Rotary Drum Thickeners and Combine Thickening and Dewatering Buildings

Similar to traditional PCs, dissolved air flotation (DAF) thickeners were recommended in TM 6 – Biosolids Treatment for process benefits while an alternative with a lower capital cost is available. There was not an evaluation conducted for WAS thickening as part of this project, but there was for the THC WWTP Expansion. The evaluation resulted in Rotary Drum Thickeners (RDTs) as the selected thickening technology. Implementing RDTs at the MCR WWTP in place of DAFs would reduce thickening equipment cost. Moreover, the RDTs could be housed with the centrifuges in a single Solids Processing Building similar to THC WWTP. Capital cost savings represent the elimination of the first and second stage DAFs, replacing the DAFs with RDTs and combining the Thickening Building with the Dewatering Building. Combing the structures increases the complexity of the building and may impact the critical path during construction. Additionally, no unthickened WAS storage is included with the change to RDTs. Unlike DAFs, RDTs would require an operator to be present at all times to allow the process to continuously waste from the secondary process; therefore, the DAF process benefit of eliminating the need for second and third shifts is removed with this VE item.

5.3.8 Combine Thickening and Dewatering Buildings

It is possible to keep the DAFs as the selected thickening treatment technology and combine the Thickening and Dewatering Buildings. The combined facility would house the centrifuges and DAF polymer feed equipment. Combing the structures increases the complexity of the building and may impact the critical path during construction. Capital costs savings represent the replacement of the Thickening and Dewatering Buildings with a two-story building that contains all thickening and dewatering equipment.

The overall project cost savings for each of these options are displayed in Table 5-4.

Table 5-4 Value Engineering Cost Options

VALUE ENGINEERING OPTION	LINE ITEM COST SAVINGS	OPCC/GMP SAVINGS	2020 OPPC SAVINGS	2031 OPPC SAVINGS
Eliminate Phosphorus Recovery	$7,138,000	$15,774,000	$18,555,000	$26,090,000
Eliminate CNG Facility	$2,912,000	$6,428,000	$7,395,000	$10,642,000
Eliminate PEW PS	$1,038,000	$2,286,000	$2,447,000	$3,792,000
°Select PCs with CEPT(1)	$2,883,000	$6,253,000	$7,187,000	$10,353,000
°^Select Disk Filters / Eliminate PSPS(1)	$4,254,000	$9,393,000	$10,939,000	$15,548,000
^Combine Headworks Building and PSPS(1)	$814,000	$1,791,000	$1,855,000	$2,973,000
*Select RDTs / Combine Thickening and Dewatering Buildings(1)	$9,363,000	$20,683,000	$24,431,000	$34,223,000
*Combine Thickening and Dewatering Buildings(1)	$3,530,000	$7,793,000	$9,027,000	$12,901,000
(1) Value engineering options with the same symbol (°, ^, or *) cannot be combined.				

5.4 PHASED CONSTRUCTION

If population/flow growth at MCR is slower than forecasted, it could be feasible to have a phased construction of the MCR WWTP Expansion to reduce capital costs of the first phase. In Figure 2-3 of TM 1 – Background, Flows, Loadings, and NPDES, a growth rate of 1.0 percent or less showed that an AA plant capacity of 15.75 mgd (or 3 out of 4 liquid trains) would be sufficient through about 2060, which is 25 years after the MCR WWTP Expansion is planned to be online. This option would have substantial cost savings for the initial project but would incur more total capital costs for JCW to build the plant out to the ultimate capacity of 21 mgd. The cost savings to the initial MCR WWTP Expansion would be associated with eliminating one PC, one BNR train, and one FC, as well as a coordinated reduction in support equipment (such as blowers and pumps). For the cost savings, no reduction in solids facilities is included. The short-term cost savings of implementing only phase one of construction is displayed in Table 5-5.

Table 5-5 Phased Construction Savings

LINE ITEM COST SAVINGS	OPCC/GMP SAVINGS	2020 OPPC SAVINGS	2031 OPPC SAVINGS
$9,655,000 (1)	$21,328,000	$25,202,000	$35,290,000

(1) Includes the elimination of one PC, one BNR train, and one FC, as well as a coordinated reduction in support equipment (such as blowers and pumps).

6.0 Summary of Operation and Maintenance Costs

Operation and Maintenance (O&M) costs were estimated and presented for the technologies recommended in previous TMs. The O&M costs presented in this section are a summary of the prior O&M costs and overall plant O&M costs. These costs do not reflect the implementation of any VE options. All estimates are in January 2020 dollars.

6.1 POWER O&M COST

The power costs in previous TMs did not include building costs associated with lighting, ventilation, and HVAC. The overall plant lighting, ventilation, and HVAC costs were estimated and included as a separate line item. All power costs were developed based on a rate of $0.073/kWh, which was provided by JCW. The total power O&M cost for the MCR WWTP is summarized in Table 6-1.

Table 6-1 Summary of Power O&M Cost

	ANNUAL O&M COST
TM 2 – Preliminary and Primary Treatment	$25,000
TM 3 – Secondary and Sidestream Treatment	$532,000
TM 4 – Auxiliary Wet Weather Treatment	$31,000
TM 5 – Disinfection Treatment	$23,000
TM 6 – Biosolids Treatment	$255,000
TM 7 – Support Facilities	$255,000
Overall Lighting, Ventilation, HVAC	$18,000
Total	**$1,139,000**

6.2 EQUIPMENT MAINTENANCE O&M COST

The equipment maintenance O&M cost was developed for each facility and included in previous TMs. The equipment maintenance cost is two percent of the equipment cost (except for the chemical feed systems, which used one percent of the total equipment cost). The total equipment maintenance O&M cost for the MCR WWTP is summarized in Table 6-2.

Table 6-2 Summary of Equipment Maintenance O&M Cost

	ANNUAL O&M COST
TM 2 – Preliminary and Primary Treatment	$49,000
TM 3 – Secondary and Sidestream Treatment	$113,000
TM 4 – Auxiliary Wet Weather Treatment	$126,000
TM 5 – Disinfection Treatment	$56,000
TM 6 – Biosolids Treatment	$285,000
TM 7 – Support Facilities	$750,000
Total	**$1,379,000**

6.3 LABOR O&M COST

Labor requirements were developed in each TM for all facilities that require operations and maintenance staff. Staff estimates assume single-shift operation due to the selected treatment processes. Additionally, there will be superintendent, supervisory, and technician staff for plant operations. It is estimated that the plant will be staffed with about 22 full-time staff members based on the following:

- Superintendents – 1
- Assistant Superintendents – 2
- Crew Members (O&M) – 14
- Electrical Technicians – 3
- HVAC Technicians – 1
- Maintenance Specialists – 1

From the MCR WWTP, staff currently services 12 pump stations in the collection system. Two additional pump stations are expected to be online by the time the MCR WWTP Expansion project is completed. Of the crew members, two to three of the positions would be dedicated to pump stations. Electrical technicians provide service for analytical instruments.

The costs are based on an estimate of yearly operations and maintenance and an hourly rate of $33.94. The total labor O&M cost for the MCR WWTP is summarized in Table 6-3.

Table 6-3 Summary of Labor O&M Cost

	ANNUAL O&M COST
TM 2 – Preliminary and Primary Treatment	$64,000
TM 3 – Secondary and Sidestream Treatment	$149,000
TM 4 – Auxiliary Wet Weather Treatment	$18,000
TM 5 – Disinfection Treatment	$18,000
TM 6 – Biosolids Treatment	$146,000
TM 7 – Support Facilities	$15,000
Superintendent, Supervisory, and Technician Staff	$1,553,000
Total	$1,963,000

6.4 CHEMICAL O&M COST

Chemical O&M costs were calculated for each process and included in previous TMs. These costs were based on the estimated chemical use per year and each chemical's cost. See TM 7 – Support Facilities for a summary of the storage and feed locations for each chemical. The total chemical O&M cost for the MCR WWTP is summarized in Table 6-4.

Table 6-4 Summary of Chemical O&M Cost

	ANNUAL O&M COST
TM 2 – Preliminary and Primary Treatment	-
TM 3 – Secondary and Sidestream Treatment	$237,000
TM 4 – Auxiliary Wet Weather Treatment	$11,000
TM 5 – Disinfection Treatment	-
TM 6 – Biosolids Treatment*	$692,000
TM 7 – Support Facilities	-
Total	**$940,000**
*Includes biosolids cake disposal costs.	

6.5 NATURAL GAS O&M COST

In addition to power, equipment maintenance, labor, and chemicals, there will be an annual O&M cost associated with natural gas demand. The natural gas demand for individual facilities was not calculated and the natural gas O&M cost was not included in previous TMs. An annual average natural gas demand of approximately 114 million cubic feet (cf) was calculated based on the plant-wide peak demand of 26,000 CFH. At a rate of $5.10/mmBTU, the total natural gas O&M cost was estimated to be $581,000.

6.6 MUNICIPAL WATER

Similar to natural gas, the municipal water demand for individual facilities was not calculated and the municipal water O&M cost was not included in previous TMs. Under normal conditions, the PEW PS will provide most service water demands. The estimated average potable demand is 60 gpm, or about 2.6 MG per month. Based on a rate of $4.24 per 1,000 gallons, the annual municipal water O&M cost is estimated to be $134,000.

6.7 O&M COST OFFSET

Finally, two of the recommended facilities on site could produce revenue that would slightly offset the annual O&M costs. The Ostara Phosphorus Recovery system produces crystalized struvite granules that can be sold on the market as Crystal Green™ fertilizer. For now, it is anticipated that the fuel produced at the CNG Facility will be used by JCW-owned trucks. Thus, the revenue accrued by the CNG Facility will be realized as a cost savings on vehicle fuel for JCW trucks. Both the Ostara and CNG processes are described in greater detail in TM 6 – Biosolids Treatment. The total revenue for the MCR WWTP is summarized in Table 6-5.

Table 6-5 Annual O&M Cost Offset

	ANNUAL REVENUE
Crystal GreenTM Fertilizer	$16,000
CNG Fuel	$193,000
Total	$209,000

6.8 SUMMARY O&M COSTS

The total annual O&M costs at the MCR WWTP are summarized in Table 6-6.

Table 6-6 Summary of MCR WWTP O&M Costs

	ANNUAL O&M COST
Power	$1,139,000
Equipment Maintenance	$1,379,000
Labor	$1,963,000
Chemicals	$940,000
Natural Gas	$581,000
Municipal Water	$134,000
O&M Cost Offset (Phosphorus and CNG)	($209,000)
Total	**$5,927,000**

7.0 Summary of Findings and Recommendations

7.1 UTILITIES

It is anticipated that there will be no need to construct a substation at the MCR WWTP. Evergy stated that the expected demand can be provided by the current substations and the cost of a new substation or an existing substation expansion would not be the County's to pay as a lump sum.

To meet the future municipal water demand, it is anticipated that a new water meter will need to be installed. The peak demand will need to be calculated closer to implementation to determine if sufficient capacity can be provided by a 4-inch or 6-inch meter. In 2020 dollars, the cost of a new 4-inch meter is $150,000, while the cost of a 6-inch meter is $350,000.

To deliver natural gas to the plant, a new natural gas service line will need to be installed. Kansas Natural Gas estimates that this addition will cost $500,000 to $750,000 in 2020 dollars. This estimate will need to be verified closer to implementation.

7.2 CONSTRUCTION PERMIT REQUIREMENTS

Multiple local, state, and federal permits are required for construction. The City of Shawnee requires a building permit, floodplain development permit, LDP, PIP and SMP along with right-of-way and temporary sign permits. Other permits include Section 404 permits (USACE), general NPDES stormwater permit (KDHE), and a floodplain and stream permit (KDA-DWR). The cost of the right-of-way, temporary sign, and building permits will be covered by the Contractor, while all remaining permits will be paid by JCW.

7.3 IMPLEMENTATION SCHEDULE

The implementation schedule was separated into four phases: engineering design, CMAR pre-construction, permitting, and construction. Engineering design and CMAR pre-construction will run concurrently along with the permitting process. Construction will consist of five primary activities: site fill & MOPO, filter complex & UV facility, construction, startup & substantial completion, and demolition & closeout. The Integrated Management Plan requires the expansion project to be online by 2035, so startup is scheduled to be completed by the end of 2034. To reach this benchmark, an Engineer will need to be selected and have an NTP by March 2028.

7.4 SUMMARY OF PROJECT CAPITAL COSTS

The OPPC was developed based on the facilities and technologies recommended in previous TMs. After factoring in site planning and other contingencies, the resulting OPPC was estimated to be $459M in 2020 dollars, or $635M in 2031 dollars (which is when construction is projected to begin). To reduce costs, several value engineering options were identified that involve combining facilities, selecting alternative technologies, and eliminating facilities that are not required to meet permit limits. One or multiple value engineering options can be implemented to reduce project costs, but some options are mutually exclusive or have downstream process impacts that affect other VE items. Another possible option for saving capital costs of the expansion project in the short-term is planning for phased construction. This would include one phase that builds the WWTP out to 15.75 mgd, followed by a second phase that expands the WWTP to the ultimate capacity of 21 mgd. Although this would increase the overall cost of the ultimate WWTP, the short-term savings is estimated to be about $25.2M in 2020 dollars (or $35.3M in 2031 dollars).

7.5 SUMMARY OF OPERATION AND MAINTENANCE COSTS

The facilities recommended in other TMs each were presented with associated power, equipment maintenance, labor, and chemical O&M costs. These summaries did not include building power, gas, municipal water, or overall labors costs for full-time staff. Including these costs, the total O&M cost for the expanded WWTP is estimated to be $5.9M per year in 2020 dollars.


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